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Public Roads

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Volume 28

April 1954-February 1956

U. S. DEPARTMENT OF COMMERCE

BUREAU OF PUBLIC ROADS

GOVERNMENT PUBLICATIONS
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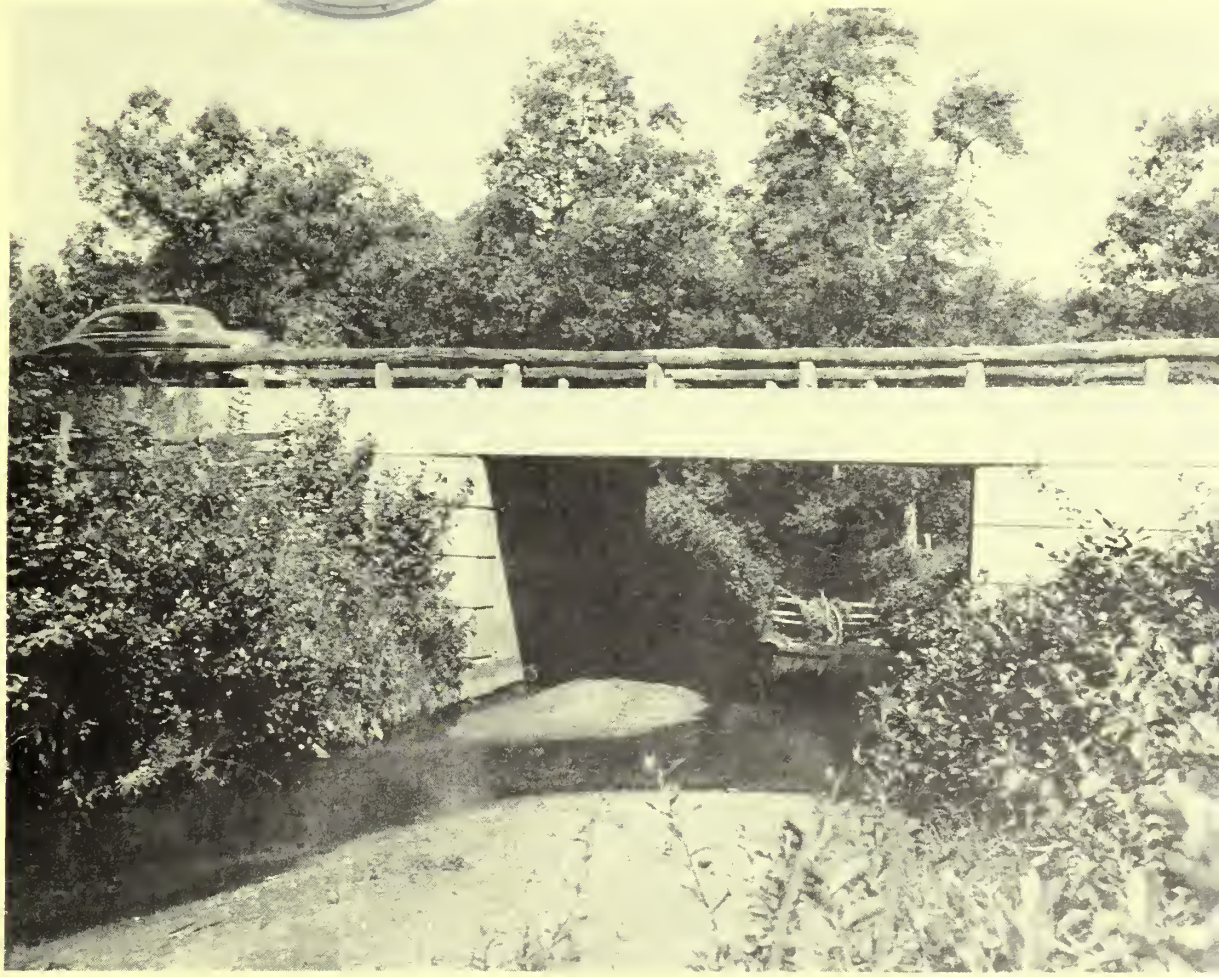

On the Blue Ridge Parkway in Virginia

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Use of Indices in Estimating Peak Rates of Runoff

BY THE HYDRAULICS RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
WILLIAM D. POTTER
Chief, Hydrology Section

Every highway bridge or drainage structure is designed for some estimated peak rate of runoff. If this estimate is too high, the cost of the structure will be excessive. On the other hand, if the estimate is too low, the cost of maintenance and replacement will be excessive. Somewhere between these extremes lies the most economical design.

Regardless of the procedures used to determine the frequency with which we can afford to have the capacity of our structures exceeded, the question arises as to how best to estimate peak rates of runoff for selected frequencies.

One of two general lines of procedure may be followed. The first of these was very popular at the beginning of the century and is still being followed by many highway departments today. It consists primarily of lumping the effects of the many variables that affect runoff into one all-embracing coefficient which is used in some unsubstantiated empirical formula to obtain a peak rate of runoff of unknown frequency. The computed peak rate is then used as the basis for the hydraulic design of our highway structures.

The second line of procedure, and the one with which this article is concerned, requires the measurement of the principal factors that affect runoff and the determination of their relations to the magnitude and frequency of peak rates. The problem of computing the size of bridge required to discharge a given flood safely once the peak runoff is estimated is beyond the scope of this article.

SINCE the turn of the century much hydrologic data have been collected by such agencies as the Soil Conservation Service, Geological Survey, and Weather Bureau. These data are now available in the form of physiographic and topographic maps, stream flow and precipitation measurements, and aerial photographs. By the use of sound statistical procedures these data may be analyzed to furnish reliable estimates of the magnitude and frequency of peak rates of runoff for use in the hydraulic design of highway structures.

It is true that the magnitude of any peak rate of runoff is the net result of innumerable hydrologic factors. Fortunately for engineers, however, the measurement of all of these many variables is not necessary to obtain reliable estimates of peak rates for selected frequencies from watersheds of a limited size range located in the same physiographic area.

For any such physiographic area and limited watershed and frequency ranges it should be possible to select three or four topographic, agronomic, or meteorologic indices that control to a considerable degree the peak rates of runoff that could be expected from watersheds within the physiographic area. It should be kept in mind, however, that these controlling indices may be different from those for any other physiographic area. Likewise, these indices may be different from those for other watersheds within the same physiographic area but differing in size ranges or in the selected frequencies of their runoff peaks. Even where the controlling indices are found to be the same for two physiographic areas, their relation to the resulting peak rate of runoff may not be the same for the two areas.

For these reasons, the relations developed for any one physiographic area should not be extrapolated beyond specified size or frequency limits nor should they be applied outside of the designated boundaries of the physiographic area. The results of statistical analyses made of peak rates of runoff from watersheds in two adjacent physiographic areas are presented here as partial verification of the foregoing statements. It is not within the scope of this article to include the detailed technical procedures [1] used, but rather to present the results of a sample study in a usable form and to demonstrate that the reliability of estimates may be greatly increased by an analysis of all available hydrologic data.

The two physiographic areas for which these analyses were made are the Allegheny-Cumberland Plateau and the Glaciated Sandstone and Shale Areas of New York, Pennsylvania, and Ohio. The relations established for both areas are applicable to mixed-cover watersheds ranging in size from 1,000 to 400,000 acres and for frequency of peak rates of runoff of from once in 10 to once in 50 years.

Conclusions

The conclusions that may be drawn from the statistical analyses of runoff from watersheds in the Allegheny-Cumberland Plateau and the Glaciated Sandstone and Shale Areas can be stated as follows:

1. The variation in the magnitude of 10-year peak rates of runoff among watersheds in a physiographic area can be satisfactorily explained by the selection of a limited number of readily attainable indices.

2. The indices that are applicable for one physiographic area are not necessarily those that explain the variations in peak rates of runoff from watersheds in some other area.

3. Even if these indices should be the same for two physiographic areas, their relation to the magnitude of peak rates of runoff would probably be different.

4. The application of the analytical procedures here developed to other physiographic areas would greatly increase the reliability of stream-flow estimates so essential for the safe and economical design of highway drainage structures.

Allegheny-Cumberland Plateau

The Allegheny-Cumberland Plateau is tentatively described by the Soil Conservation Service of the U. S. Department of Agriculture as follows:

The Allegheny-Cumberland Plateau, occupying 48,708,500 acres, is on the western slope of the Appalachian uplift. It extends westward from the Allegheny-Cumberland front where it passes through central Pennsylvania, western Maryland, eastern West Virginia, eastern Kentucky, eastern Tennessee, and north-central Alabama to the western edge where it merges into the central basin of Ohio and the Highland Rim of Kentucky, Tennessee, and Alabama. The eastern edge of the Plateau top is 2,500–3,500 feet above sea level with a few ridges or mountains extending to 3,800–4,000 feet. The western part is 1,000–1,500 feet above sea level. The Plateau along the eastern edge has somewhat flattened tops with deep narrow stream gorges. The western part is severely dissected into comparatively narrow ridges and V-shaped valleys along which the terrace and bottom lands have a limited development. The relief is gently undulating or rolling to hilly and steeply sloping with little flattish relief. It has a typical dendritic drainage pattern. The rock formations are mainly gray alternating beds of acid shale and sandstone with some thin-bedded limestone and calcareous shale of Carboniferous Age. These formations are resting in a clear-horizontal position. The slope of the formations is so gradual that except for a few minor anticlines and synclines it is not noticeable. The land is used as follows: 18.5 percent, cultivated;

[1] *Rainfall and topographic factors that affect runoff*, by W. D. Potter. Transactions of American Geophysical Union, vol. 34, No. 1, pp. 67-73, Feb. 1953.

15 percent, grassland; 50 percent, woodland; 8 percent, miscellaneous; and about 8.5 percent is in public ownership.

The climate is cool, temperate, and humid. Winters are cold; summers mild. Rainfall ranges from 40 to 50 inches. The frost-free season is 120 to 150 days in the high plateau, and 140 to 170 days from the northern to southern part. Due to steep relief the runoff is rapid in spite of the

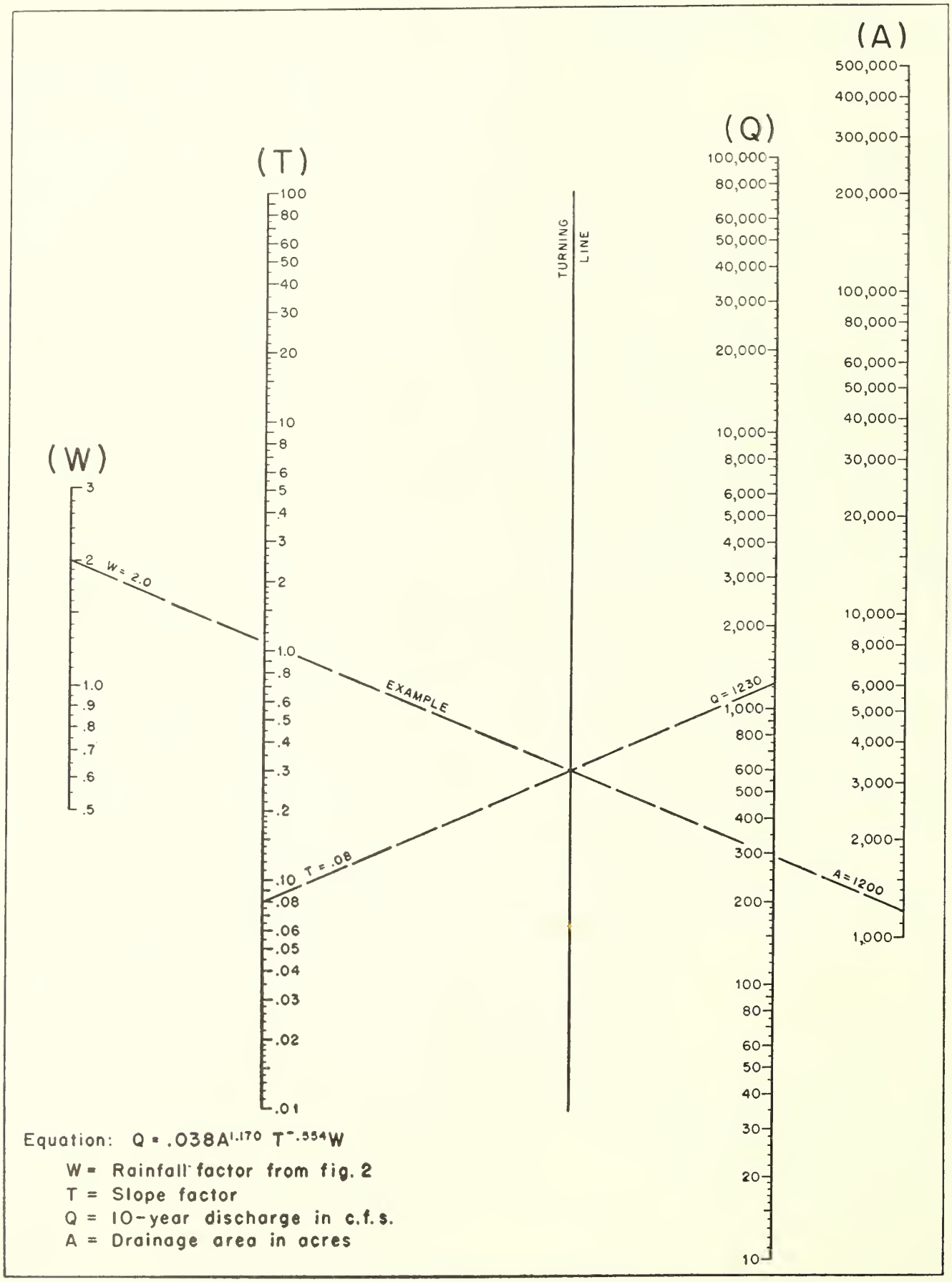


Figure 1.—Nomograph for discharge in the Allegheny-Cumberland Plateau.

large acreage of forest and grass cover. Original forest cover was oak, hickory, walnut, poplar, and maple with ash, beech, birch, and hemlock in ges, spruce on high elevations, and pitch pine n southern reaches. A fairly large acreage is in National and State forests.

The soils are residual. Under forest cover they have a thin mat of organic matter on the surface more or less mixed at the bottom with mineral soil materials. This rests on brown to gray-brown mellow soil which passes at about 8 inches into a yellow-brown friable subsoil, passing at 24 to 48 inches into partly disintegrated parent material. Over extensive areas bedrock comes within the 3-foot soil profile. Soils contain much shaly, channery, and flaggy pieces of rock materials, and in places a noticeable amount of stone, usually sandstone.

Although there is still a considerable area in forest and wood lots, much of the land is cleared and used for general farming, dairying, stock-raising (beef cattle and sheep), and orcharding (apples). Under this system of farming, a large percentage of the land is in pasture. Crop yields are fairly good where manure, lime, and fertilizers are used. These practices are in common usage. The climatic differences from north to south in the stretch of some 800 miles are responsible for variations in the agriculture of the area.

Standard Error of Estimate

Simple correlation

For the area thus described by the Soil Conservation Service, peak-rate probability curves were computed for 51 mixed-cover watersheds ranging in size from 1,000 to 400,000 acres, based on runoff measurements of the U. S. Geological Survey and the Soil Conservation Service. The runoff record available for the majority of these watersheds was for the period 1938–48, and this short record was therefore selected for the computation of the probability curves. The average relation was determined between the 10-year peak rates and the corresponding watershed area. A statistical analysis showed that 67 times out of 100 the 10-year peak could be expected to vary from that indicated by this relation by as much as 35 percent. In other words, the standard error was 35 percent. This large standard error was evidence that the variation of 10-year peak rates could not be satisfactorily explained by consideration of area alone.

Multiple correlation

Three additional variables were selected, two of them indicative of the precipitation experienced during the short period of runoff record and the other indicative of topographic features. The precipitation indices were designated as the P and S ratios and were computed from U. S. Weather Bureau records for the period 1938–48. The P ratios were numerical expressions of the differ-

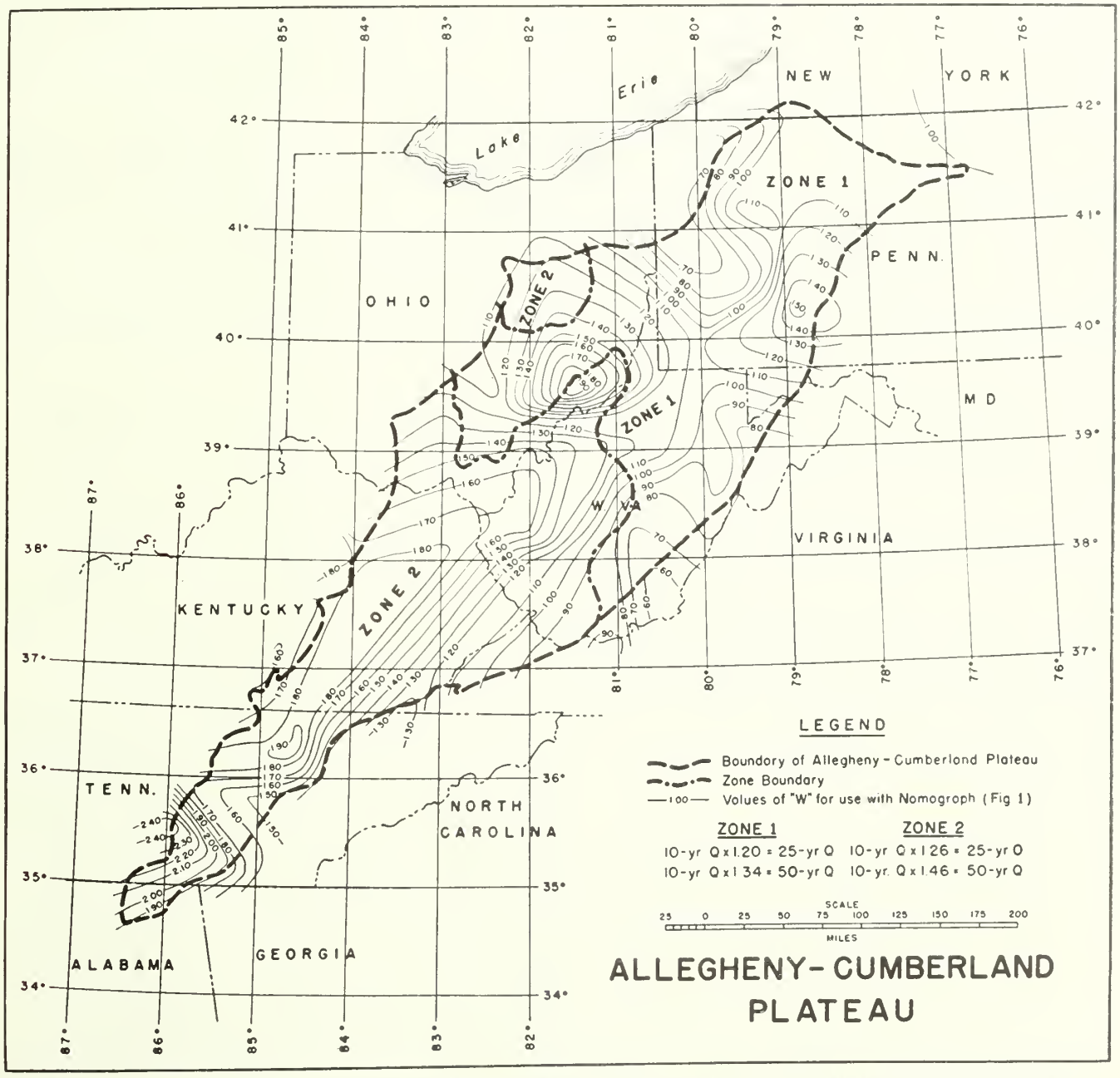


Figure 2.—Values of W for use with the nomograph (fig. 1) for discharge in the Allegheny-Cumberland Plateau.

ences in the intensity of precipitation among watersheds and the S ratio expressed the difference in annual precipitation and the number or frequency of excessive storms. The topographic index was designated as the T factor. It was computed from the topographic maps of the Geological Survey and when considered with area was indicative of differences in watershed shape, channel storage, and time of concentration.

A multiple correlation was then computed in which the watershed area A, the P ratio, the S ratio, and the T factor were considered as independent variables with the 10-year peak rate of runoff (q) as the dependent variable. A statistical analysis of the relations determined by this correlation showed that the standard error was 18 percent. In other words, the reliability with which 10-year peaks could be estimated 67 times out of 100 had been almost doubled over what it had been when area alone was considered. The progressive reduction in standard error as each new variable was introduced into the correlation is as follows:

	Standard error in percent
q vs. A	35
q vs. A and T	30
q vs. A, T, and P	21
q vs. A, T, P, and S	18

The 10-year peak rates used in this study were all predicated on short periods of runoff records. They could be indicative of future events, therefore, only if the factors that affected runoff during that short period were representative of a much longer period. Since the topographic factors of any watershed may be considered constant, the only factor affected by the length of the period of record would be precipitation. The precipitation indices P and S were, therefore, recomputed from Weather Bureau records covering the longer period 1915–48. The relations established for the short runoff period, as expressed by the regression equation, could then be utilized with these long-term precipitation indices to obtain new values of 10-year peak rates. These new values would be the peak rates that could have been expected from probability curves if the longer runoff record had been available.

Aids in Computing Peak Rates

To facilitate the solution of the regression equation a nomograph (fig. 1) and map (fig. 2) were prepared in which the two precipitation indices were combined in a single index W. To use these aids in estimating the peak rates of runoff that may be expected for a 10-, 25-, or 50-year recurrence interval, the procedure is as follows:

Area A.—Determine the area of the watershed in acres.

Precipitation index W.—Locate the watershed on figure 2 and select a precipitation index W that is the average value for that watershed.

Topographic index T.—From a U. S. Geological Survey topographic map, or any other accurate contour map of the watershed, measure the length in miles of the principal stream from the site of the proposed culvert or bridge to the headwater. The headwater, or uppermost point on the stream, should be taken as the point where a definite channel begins regardless of whether or not streamflow at this point is intermittent. If a U. S. Geological Survey map is used, the length of stream should include that portion shown as a broken blue line. Divide the length of the principal stream into two reaches, the lower reach being 0.7 of the total length and the upper reach the remaining 0.3. From the contours, determine the elevation of the stream channel at the upper and lower limits of each reach. Compute the average slope of each reach as the fall of the stream channel in feet divided by the length of channel in miles. Divide the length of stream channel for each reach measured in miles by the square root of the corresponding slope, and add the quotients. This sum is the topographic index T.

Use of nomograph.—Using the nomograph (fig. 1) and a straightedge, connect area A with rainfall factor W and mark the intersection with the turning line. Connect this intersection with the topographic factor T and extend the line to the Q scale. The intersection with the Q scale is the desired peak rate of runoff in cubic feet per second, for a 10-year recurrence interval (see example, fig. 1).

Peak rates for 25- and 50-year frequencies.—To obtain the peak rate of runoff for a 25- or 50-year frequency, multiply the 10-year peak by the corresponding coefficient (fig. 2).

Glaciated Sandstone and Shale Area

The Glaciated Sandstone and Shale Area is tentatively described by the Soil Conservation Service as follows:

The northern Appalachian Plateau or glaciated portion of the Appalachian Plateau is situated in southern New York, northern Ohio, and northern Pennsylvania. The plateau top, consisting of nearly level plains and rolling hills, ranges from 1,200–

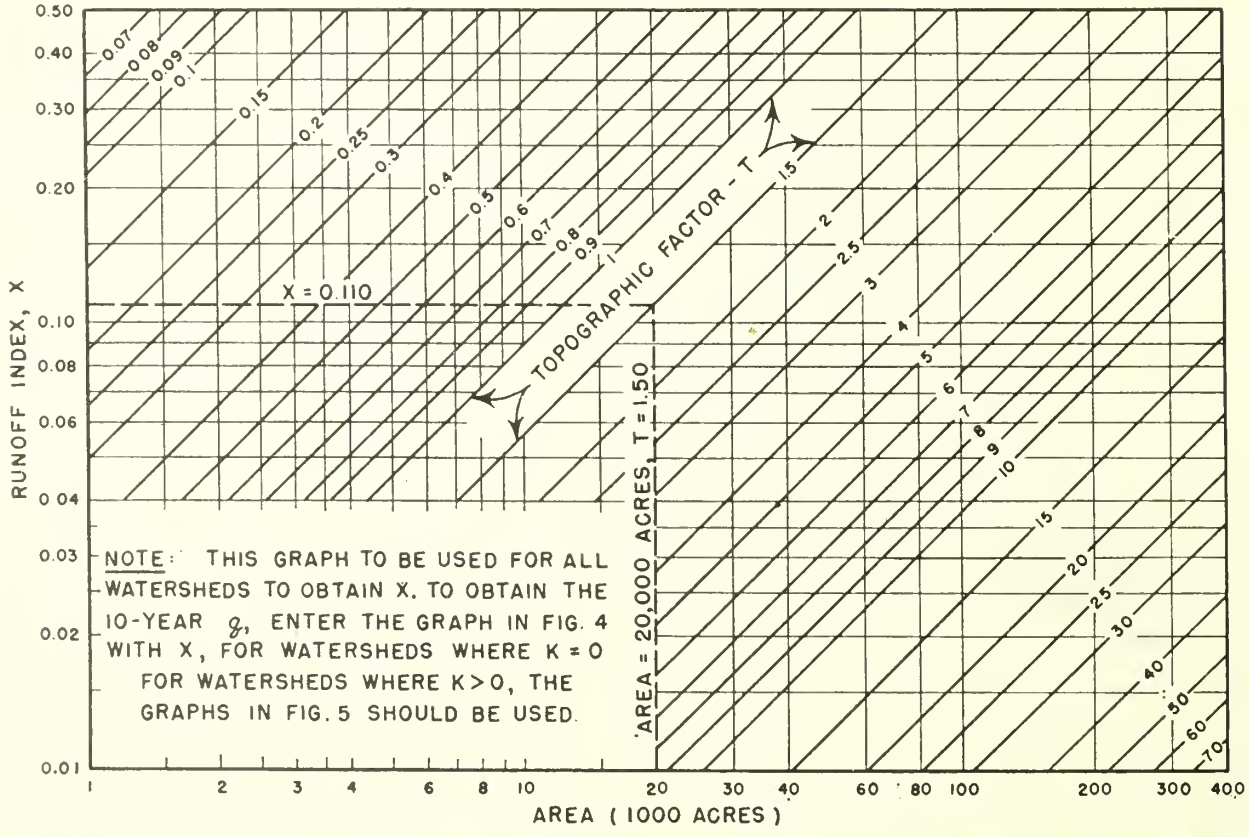


Figure 3.—Relation between area, topography, and runoff index for the Glaciated Sandstone and Shale Area.

1,500 feet in the western part to 1,800–2,500 feet in the eastern part. The geologic material consists of acid gray sandstone and shale over much of the area, with red sandstone and shales in the eastern part, and mixed alkaline and acid gray shales and sandstones, with some limestone material along the northern border. This is covered by drift from the Wisconsin glaciation consisting of bedrock material with little influence from outside sources.

This is essentially a "double-deck" country, consisting of a high plateau flattened to undulating to rolling, having fairly steep valley walls and fairly broad valleys filled with till and outwash material. The valley floors are about 1,000–1,200 feet below the general plateau level. A considerable area of the higher plateau is in forest and a lesser area of the remainder is forested.

The climate is cool, temperate, and humid. Winters are long and cold. Precipitation ranges from 35 to 50 inches from west to east. Frost-free period on the plateau top is 110 to 130 days and in the valleys about 2 to 3 weeks longer.

The soils on the higher plateau are moderately deep to shallow and at lower elevations they are somewhat deeper. Soils are generally medium-textured, with moderately heavy subsoils, and are mainly acid with some alkaline and calcareous materials. Some areas are poorly drained, but mainly they are excessively drained. The conservation problem is moderate to severe water erosion, but the main problem is controlling erosion on steep slopes. This is done mainly by strip cropping, permanent pasture, or reforestation. The land is used for general farming, dairying, and fruit growing. On the higher plateau the crops are hay, oats, buckwheat, and potatoes, and on the lower areas hay, corn, wheat, oats, potatoes, vegetables, and fruit. There is much abandoned land on the plateau, some of which is being reforested by Federal and State agencies, and some is being purchased for potato production and other private enterprise.

The total area is 23,550,000 acres. Of this, 1,325,000 acres or 5½ percent is publicly owned, usually in State forest and parks; 2,151,000 acres or 9 percent is miscellaneous; 3,768,000 acres or 16 percent is in private forest; and 15,909,000 acres or 68 percent is in farms. Of the land in farms, 46 percent is cropland; 26 percent is grassland (pasture); 20 percent is woodland; and 6 percent is all other farm land.

Relation of Two Variables

Following the same general procedure for the Glaciated Sandstone and Shale Area, thus described by the Soil Conservation Service, as that used in the case of the Allegheny-Cumberland Plateau, frequency curves were computed for the peak rates of runoff from 47 mixed-cover watersheds gaged by the U. S. Geological Survey and ranging in size from 1,000 to 400,000 acres. These probability curves were based on the short period of available runoff records starting with 1938 and extending through 1950. A statistical analysis of the average relation between the 10-year peak rate and the corresponding watershed area showed a standard error of 46 percent: that is, 67 times out of 100, the 10-year peak could be expected to vary from that indicated by this relation by as much as 46 percent. This large standard error led to the same conclusion as that made in the Allegheny-Cumberland study: that the variation of the 10-year peaks among watersheds could not be satisfactorily explained by consideration of area alone.

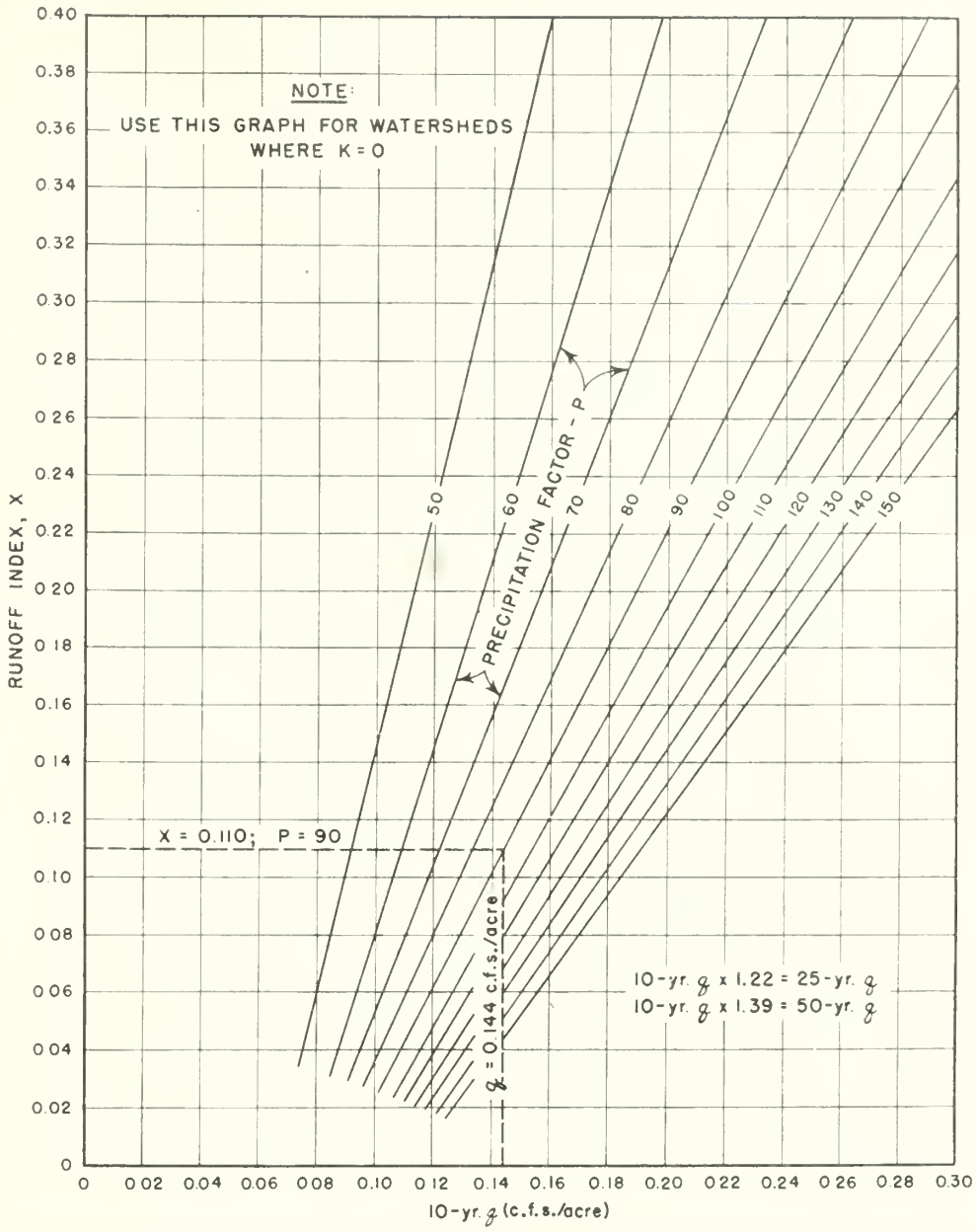
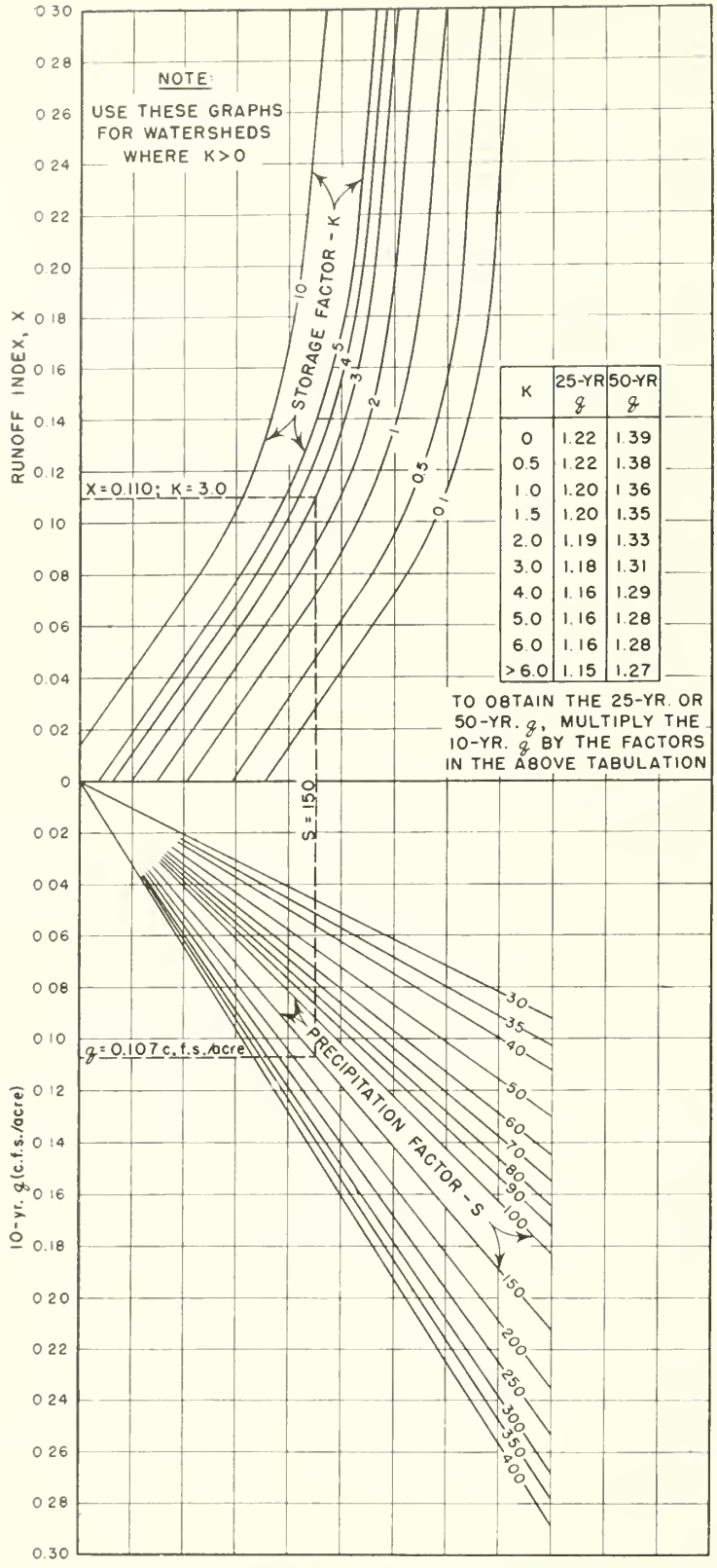


Figure 4.—Relation between runoff index, precipitation, and 10-year peak rate of runoff for the Glaciated Sandstone and Shale Area.

Additional Factors Considered

The two precipitation indices P and S, and the topographic index T, as described for the Allegheny-Cumberland Plateau, were computed for the Glaciated Sandstone and Shale Area. The precipitation indices were based on Weather Bureau records and were indicative of the precipitation experienced during the short period of runoff record. In addition, a new index K was selected to express the difference in runoff storage among watersheds. This storage was defined as the temporary or permanent storage occasioned by the presence of swamps, lakes, or reservoirs within the watershed or by the overflow of the lower reaches of the stream over all or part of its flood plain.

Graphical correlations (figs. 3–5) were made for the addition of each of these indices in which the 10-year peak rate (q) was taken as the dependent variable and the area of the watershed and the various indices as the independent variables. The standard error was computed for each correlation. For the 14 watersheds that had no runoff storage, it was found that the introduction of the topographic index T and the precipitation index P reduced the standard error of the correlation to 18 percent. No additional reduction was obtained by the further addition of the second precipitation index S. For the 33 watersheds that had runoff storage, however, the effect of the precipitation indices was reversed. The introduction of the topographic and storage indices T and K and the precipitation index S reduced the standard error to 18 percent, but no additional reduction was obtained by the further addition of the precipitation index P.



K	25-YR q	50-YR q
0	1.22	1.39
0.5	1.22	1.38
1.0	1.20	1.36
1.5	1.20	1.35
2.0	1.19	1.33
3.0	1.18	1.31
4.0	1.16	1.29
5.0	1.16	1.28
6.0	1.16	1.28
>6.0	1.15	1.27

Figure 5.—Relation between runoff index, storage, precipitation, and 10-year peak rate of runoff for the Glaciated Sandstone and Shale Area.

As in the Allegheny-Cumberland Plateau, the reliability with which 10-year peaks could be estimated was materially increased by the consideration of topographic and precipitation indices. In the Glaciated Sandstone and Shale Area this increase in reliability was two-and-a-half times what it had been when area alone was considered. The progressive reduction in standard error as each new variable was introduced into the correlation is as follows:

	Standard error in percent
q vs. A	46
q vs. A and T	41
q vs. A, T, and K	24
q vs. A, T, K, and P or S	18

The relations established by the correlation graphs (figs. 3–5) for the short period of runoff record were utilized to obtain 10-year peak rates that would be representative of much longer periods. To do this it was necessary to recompute the precipitation indices from Weather Bureau records for the longer period 1915–50. Maps showing the areal variations of these long-term P and S indices (fig. 6) were prepared for use with the correlation graphs.

Maps and Graphs Utilized

To use these maps and correlation graphs in estimating peak rates of runoff that may be expected for a 10-, 25-, or 50-year recurrence interval, the procedure is as follows:

Area A.—Determine the area of the watershed in acres.

Topographic index T.—From a U. S. Geological Survey topographic map, or any other accurate contour map of the watershed, determine the topographic index T in the manner outlined for the Allegheny-Cumberland Plateau.

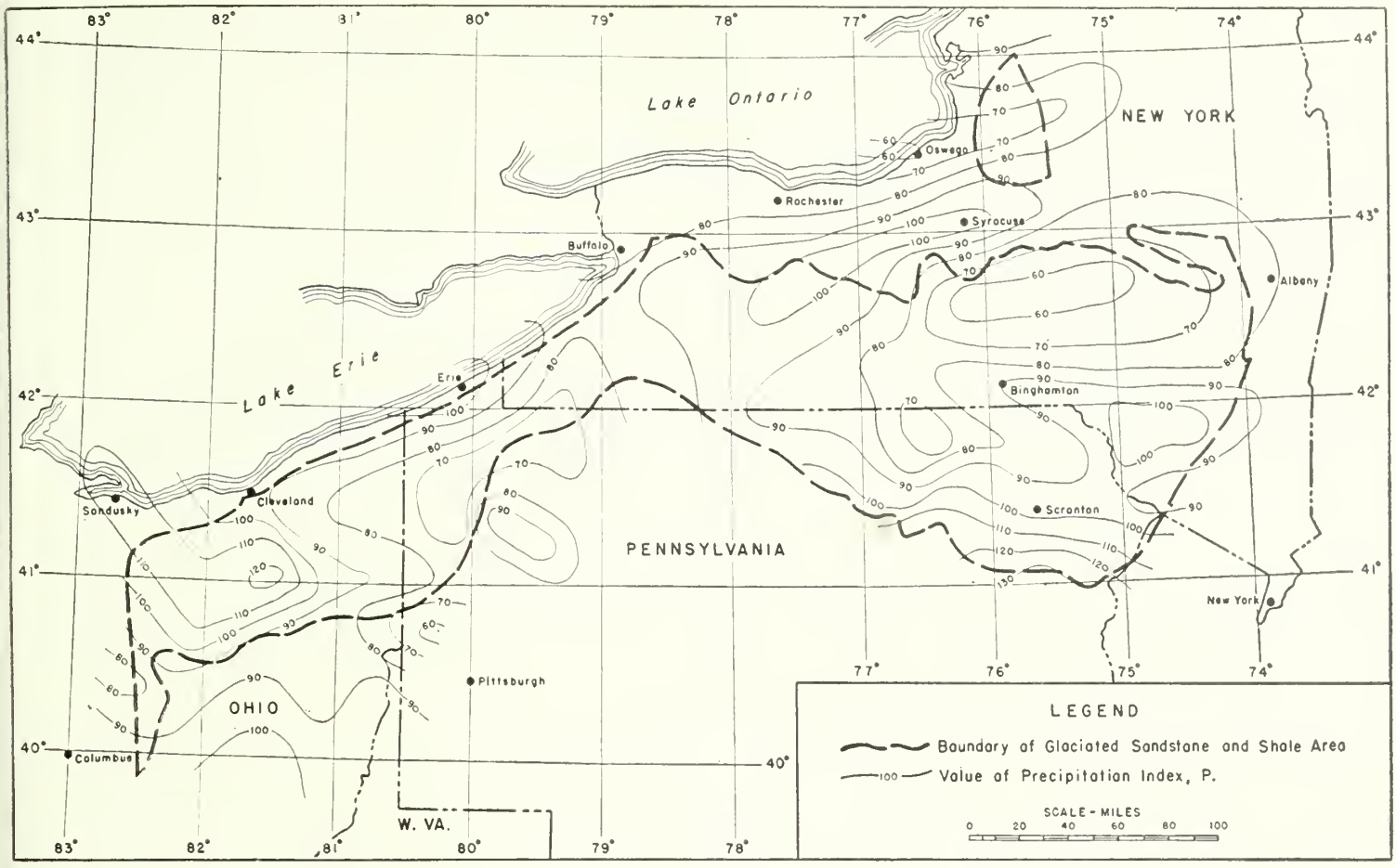
Storage index K.—From a U. S. Geological Survey topographic map, or any other accurate contour map of the watershed, measure the area of all swamps, lakes, or reservoirs. Also, where the average width of the valley bottom exceeds a quarter of a mile, a field inspection should be made to determine whether or not flooding of the valley is likely to occur. When flooding appears likely, a determination of the area that may be assumed to be covered by a 10-year flood should be made from the topographic map. For the purpose of use with the correlation graph (fig. 5), this flooded area should be taken as the total area of the valley bottom where its average width exceeds one quarter of a mile. Where railroads or major highways have been constructed on the valley bottom, it may be assumed that their embankments limit the extent of lateral flooding and they may be taken as one boundary of the flooded area. The sum of the areas of all swamps, lakes, reservoirs, and flood plains divided by the area of the watershed expressed in the same unit is the storage index K. (In measuring the areas, a considerable saving of time results from tracing the boundaries of these areas on tracing paper as one composite area which can then be planimetered in one operation.)

Precipitation index P.—If the storage index K for the watershed is zero, locate the watershed on figure 6 and select a precipitation index P that is the average value for that watershed.

Precipitation index S.—If the storage index K for the watershed is greater than zero, locate the watershed on figure 6 and select a precipitation index S that is the average value for that watershed.

Use of graphs.—To determine the 10-year peak rate of runoff, enter figure 3 with the watershed area and move vertically up the graph to the value on the T scale corresponding to that computed for the watershed. Move horizontally across the graph to the ordinate scale and record the indicated runoff index.

If the value of K for the watershed is zero, enter figure 4 with the runoff index and move horizontally across the graph to the value on the P scale corresponding to that selected for the water-



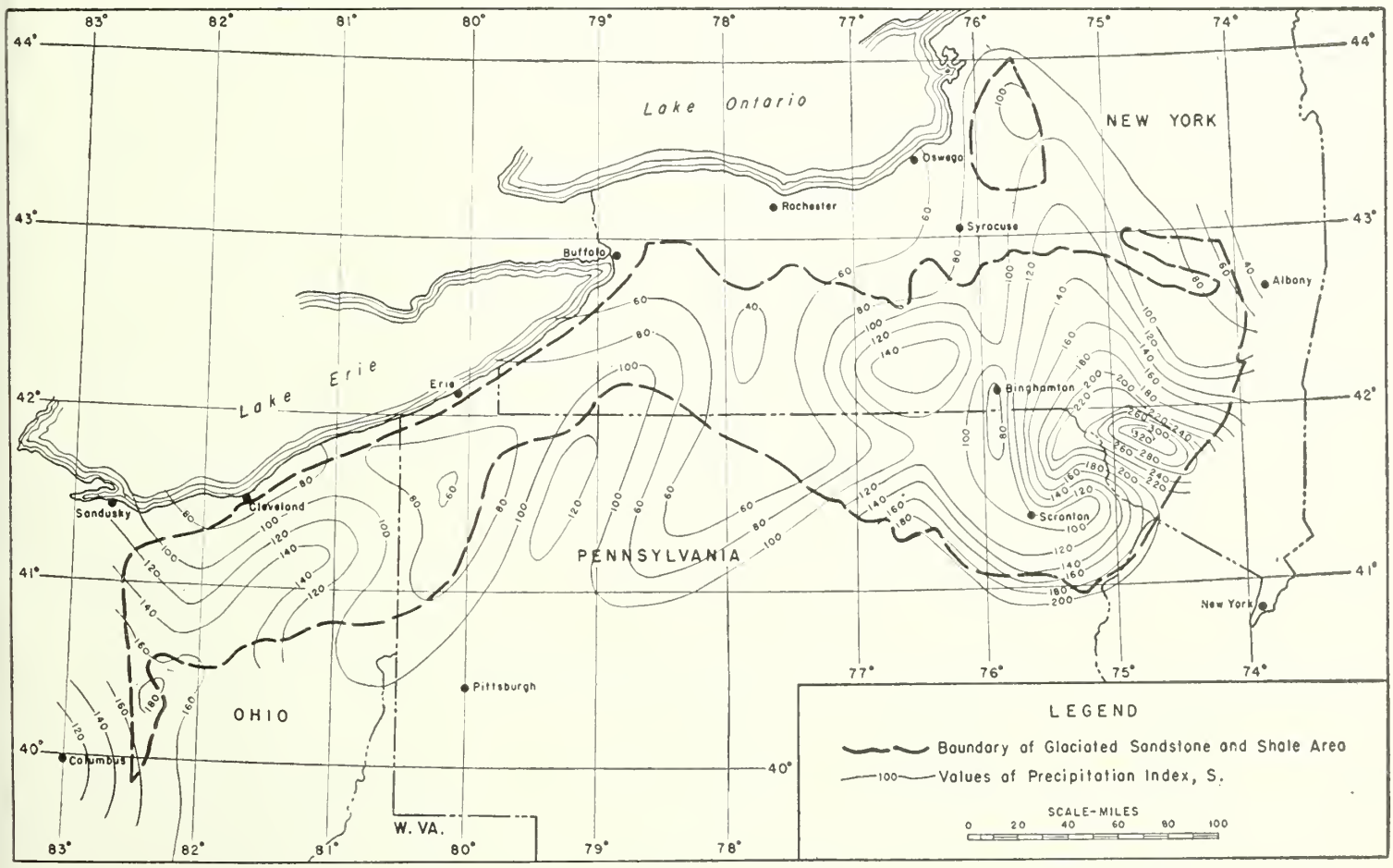


Figure 6.—Precipitation factors P (above) and S (below) for the Glaciated Sandstone and Shale Area.

shed. Move vertically down the graph to the abscissa q_{10} scale and record the indicated 10-year peak rate of runoff in cubic feet per second per acre. Multiply this value by the area of the watershed in acres to obtain the 10-year peak in cubic feet per second.

If the value of K for the watershed is greater than zero, enter figure 5 with the runoff index and move horizontally across the graph to the value on the K scale corresponding to that computed for the watershed. Move vertically down the graph to the selected value of S as read on the S scale and thence horizontally across the graph to the ordinate q_{10} scale. Record the indicated 10-year peak rate of runoff in cubic feet per second per acre and multiply this value by the area of the watershed in acres to obtain the 10-year peak in cubic feet per second.

Peak rates for 25- and 50-year frequencies.—For watersheds where K is zero, multiply the 10-year peak rate by 1.22 to obtain the 25-year peak or by 1.39 to obtain the 50-year peak (fig. 4). For watersheds where K is greater than zero, select from the tabulation on figure 5 coefficients for 25- and 50-year peaks corresponding to the value of K computed for the watershed. Multiply the 10-year peak rate by these coefficients to obtain the 25- or 50-year peak.

Reliability of Estimates

It has already been shown that 67 times out of 100 the 10-year peak rates of runoff from watersheds within the two physiographic areas here considered may be expected to vary from estimates derived through the use of the nomograph (fig. 1) or graphs (figs. 3–5) by not more than 18 percent. This degree of reliability of the estimates may be considerably increased by the exercise of judgment on the part of the design engineer. The descriptions of the physiographic areas are descriptions of the topography, soils, or cover that apply to the average watershed within the area. These factors may vary considerably from the average for any individual watershed. The design engineer should think of the values derived from the graphs as being bounded by 18-percent upper and lower bands within which he is to exercise his own judgment. If, for instance, an inspection of a watershed shows the depth (and hence the water-holding capacity) of the soil to be considerably less than that described as average, he might be justified in increasing the value obtained from the graphs by as much as 18 percent. Conversely if the soils were found to be deeper or more porous than described for the average, he might reduce the graph peaks by a like percentage.

Design of Channel Shear Connectors for Composite I-Beam Bridges

Reported by[1] IVAN M. VIEST, Research Assistant Professor of Theoretical and Applied Mechanics, and

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In a composite I-beam bridge, a rigid connection between the concrete slab and the steel beams increases the stiffness of the bridge and permits use of smaller beams than would otherwise be required. The connection has a two-fold purpose—to prevent movement between the slab and the beams, and to transfer horizontal shear from one element to the other. Such a connection may be obtained by the use of channels welded to the top flanges of the beams and embedded in the concrete.

In this article a procedure for the design of flexible channel shear connectors for composite I-beam highway bridges is developed from data obtained in studies conducted at the University of Illinois. Equations are provided for use in the case of construction with shoring and also in the case of construction without shoring. The procedure is illustrated by an example.

BRIDGES composed of steel I-beams supporting a reinforced concrete slab may be built either with the slab resting freely on, or rigidly connected to, the beams. A rigid connection between the slab and the beams will increase considerably the stiffness of the bridge and, if considered in the design, will permit the use of a smaller I-section than would be required for a noncomposite design (ref. *3*).[2] In such a composite bridge the external loads are carried both by the beams and by the slab acting as a heavy flange of the steel beams. The necessary connection between the beams and the slab may be obtained by means of mechanical shear connectors attached to the top flange of the beams and embedded in the concrete of the slab. The twofold purpose of this connection is to prevent movement between the slab and the steel beams and to transfer horizontal shear from one element to the other.

Various types of shear connectors have been proposed and used. In general, they may be divided into three categories:

1. Rigid connectors such as square steel bars attached to the top flange of the I-beams;

2. Flexible connectors such as steel channels attached to the top flange of the I-beams;

3. Bond connectors such as reinforcement bars attached to the top flange of the I-beams in the form of loops or longitudinal helixes.

The flexible connector, made of a short piece of rolled steel channel with one flange welded to the beam and the other embedded in the slab, has been the subject of extensive experimental studies conducted at the University of Illinois in cooperation with the Illinois State Division of Highways and the Bureau of Public Roads (refs. *4, 6*). Numerous tests of individual connectors, as well as tests of composite beams with a series of connectors, have been made to obtain qualitative and quantitative data on the behavior of the connectors and on the influence of variations in the dimensions of the channel and the properties of the concrete on the load-carrying capacity of channel connectors.

In this article a procedure for the design of flexible channel shear connectors for composite I-beam highway bridges is developed on the basis of the data obtained from the Illinois tests. The procedure is illustrated by a numerical example on page 16.

Considerations for Design

Behavior of Composite T-beams

The advantages of a composite concrete and steel T-beam as compared to a noncomposite beam of the same dimensions are twofold: First, at a given load the deformations and therefore also the stresses in the composite structure are smaller; second, the ultimate load for the composite beam and the deflection at which the ultimate load is reached exceed substantially those for the noncomposite beam—in other words, the composite structure is tougher. The advantages of composite action are fully realized only if the shear connection is strong enough to prevent large slip between the I-beam and the slab and thus to insure full composite action at all levels of loading, up to the ultimate load. If, to the contrary, the shear connection permits large slip at the contact surfaces of the beam and the slab, the composite action is incomplete, the deformations at a given load are larger than in a fully composite structure of the same dimensions, and the ultimate load-carrying capacity is smaller. (Supporting experimental evidence can be found in refs. *4, 6F*.)

In order to insure full benefit of the composite action, the design procedure presented in this article is based on the requirement of full composite action at all levels of loading. Such a procedure requires a knowledge of the ultimate flexural capacity of a composite T-beam.

The behavior of a composite steel and concrete T-beam subjected to a continuously increasing load may be divided into three stages. During the first stage, both the beam and the slab are elastic. The strain distribution and the stress distribution are approximately linear throughout the beam. The end of this stage is reached when the maximum stress in the beam reaches the yield-point value. During the second stage of loading, the yielding of the beam spreads upward into the web toward the top flange and along the bottom flange toward both supports, and the stresses in the slab enter the plastic range. The load increases throughout this stage of loading and the deformations increase at an increasing rate. At the end of this stage, the concrete slab fails by crushing in compression and the load-carrying capacity of the T-beam is thereby decreased considerably. If the load is allowed to decrease, the beam reaches a new equilibrium at a substantially lower load, and the third stage of loading follows.

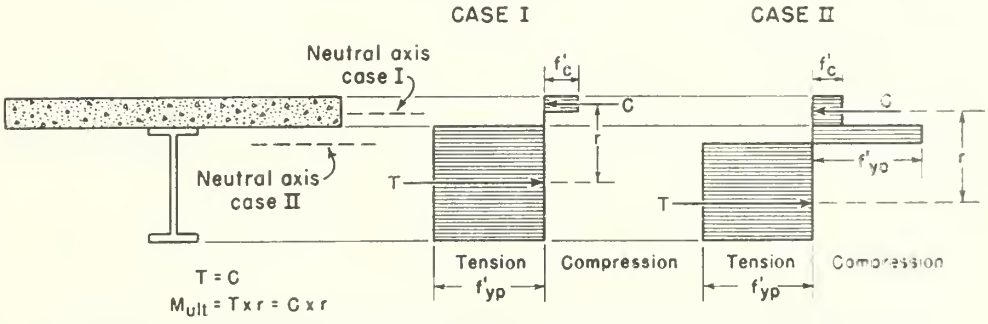


Figure 1.—Idealized distribution of stress in composite T-beam at ultimate load.

[1] This article is a report of an investigation conducted by the Engineering Experiment Station of the University of Illinois in cooperation with the Illinois State Division of Highways and the Bureau of Public Roads.

[2] Italic numbers in parentheses refer to the list of references on page 15.

NOTATION

A_c = Cross-sectional area of the portion of the concrete slab considered to act with the I-beam (*in.*²).

A_w = 0.707 weld size × weld length; effective area of the weld (*in.*²).

E_c = modulus of elasticity of concrete (*p.s.i.*).

E_s = modulus of elasticity of steel (*p.s.i.*).

$C_M = \frac{M_{DL}}{M_{LL}}$; $C_S = \frac{S_C}{S_I}$; $C_V = \frac{V_{DL}}{V_{LL}}$.

f'_c = compressive strength of concrete determined from the tests of 6 × 12-in. cylinders (*p.s.i.*).

f_{co} = maximum pressure on concrete adjacent to a channel shear connector (*p.s.i.*).

f_{max} = maximum steel stress in the web of a channel connector (*p.s.i.*).

f_s = allowable steel stress (*p.s.i.*).

f_{yp} = yield-point stress of steel in channels (*p.s.i.*).

f'_{yp} = yield-point stress of steel in beams (*p.s.i.*).

h = maximum thickness of channel flange (fig. 2) (*in.*).

h' = minimum thickness of channel flange (fig. 2) (*in.*).

I = moment of inertia of a composite T-beam computed from the area of the steel beam and the transformed area of the slab, A_c/n (*in.*⁴).

$k_1 = \frac{1.6}{\frac{f_{co}}{f'_c} + 1.8}$.

M_{ult} = resisting moment of a composite T-beam at ultimate load (fig. 1) (*in.-lb.*).

M_{DL} = maximum moment caused by the dead load (*in.-lb.*).

M_{LL} = maximum moment caused by the design live load, including impact (*in.-lb.*).

$n = E_s/E_c$.

Q = load on shear connector (*lb.*).

Q_{des} = design load on shear connector (*lb.*).

Q_{yp} = load on shear connector corresponding to first yielding of the connector (*lb.*).

Max. Q = the numerically largest load on a connector under moving design load (*lb.*).

Min. Q = the numerically smallest or the largest load of opposite direction to that of Max. Q on a connector under moving design load (*lb.*).

s = spacing of shear connectors (*in.*).

S_C = section modulus of a composite T-beam computed from the area of the steel beam and the transformed area of the slab, A_c/n (*in.*³).

S_I = section modulus of a steel I-beam (*in.*³).

S_{pl} = M_{ult}/f'_{yp}; plastic section modulus of a composite T-beam (corresponding to stress distribution shown in fig. 1) (*in.*³).

t = thickness of the web of a channel shear connector (fig. 2) (*in.*).

V = vertical shear on a composite T-beam (*lb.*).

V_{des} = maximum vertical shear on a composite T-beam at any point along the beam caused by the design load (*lb.*). In computing the design load for shear connectors (equation 7), $V_{des} = V_{LL}$ should be used for composite beams built without shoring and $V_{des} = V_{DL} + V_{LL}$ should be used for composite beams built with shoring.

V_{ult} = maximum vertical shear at any point along the beam caused by the ultimate flexural load (*lb.*).

V'_{ult} = maximum vertical shear on a composite T-beam at any point along the beam caused by the portion of the ultimate flexural load which contributes to the load acting on shear connectors (*lb.*). For a T-beam built without shoring, $V'_{ult} = V_{ult} - V_{DL}$; and for a T-beam built with shoring, $V'_{ult} = V_{ult}$.

V_{DL} = vertical shear at any point of the bridge caused by the dead load (*lb.*).

V_{LL} = maximum vertical shear at any point of the bridge caused by moving design live load, including impact (*lb.*).

w = length of a channel shear connector (*in.*).

y_c = distance between the center of gravity of the concrete slab and the neutral axis of the composite section (*in.*).

The beam carries the maximum load at the instant of crushing of the slab. The corresponding moment is called, in this article, the ultimate moment-carrying capacity of the T-beam. It has been shown by comparisons with the results of tests that the ultimate moment and load may be computed approximately on the assumption of the idealized distribution of stress shown in figure 1 (ref. *6G*). At this stage, the neutral axis of the composite section may be located either in the slab, case I in figure 1, or in the I-beam, case II in figure 1. It is assumed in both cases that the compressive stresses in the slab are uniformly distributed and equal to the concrete cylinder strength f'_c, and the tensile stresses in the steel I-beam are uniformly distributed and equal to the yield point stress f'_{yp}. It is assumed further in case I that the concrete slab cannot carry any tension, and in case II that the compressive stresses in the steel I-beam are distributed uniformly and are equal to the yield-point stress f'_{yp}. The effects of the reinforcement in the slab are neglected in both cases. With the aid of these assumptions, the position of the neutral axis may be determined from the condition of equilibrium of horizontal forces, and the ultimate moment-carrying capacity may be found from the condition of the equilibrium of moments (fig. 1).

Behavior of channel shear connectors

A cross-section of a flexible channel shear connector is shown in figure 2. The lower flange of the channel is connected rigidly to the I-beam and the entire connector is embedded in the concrete of the slab. When the connector is loaded, the lower flange, because of the rigid connection and stiffness, can neither bend nor rotate independently of the supporting beam. The flexible web deflects as a dowel. Accordingly, the distribution of pressure on the adjacent concrete is uniform adjacent to the stiff flange and nonuniform adjacent to the flexible web. The pressure distribution adjacent to the connector is shown in figure 2 as a dashed line. It can be seen that the pressure is concentrated at the end attached to the beam and that the magnitude of the pressure decreases rapidly with the distance from this end. At the upper end of the channel, both the pressure and the deflection or rotation of the web are usually so small that the conditions at this end may be neglected in discussing the behavior of a channel connector.

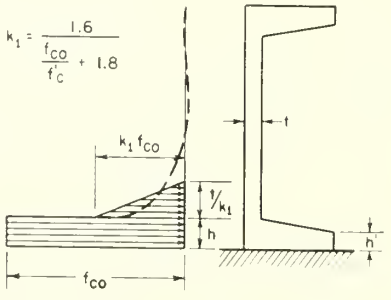


Figure 2.—Distribution of load on channel.

When the load is applied to a channel connector, the connector exerts pressure on the adjacent concrete. The connector is deformed and the concrete is compressed. The result of compressing the concrete is a relative movement between the slab and the connector and, in turn, a movement between the slab and the beam. The pressure distribution diagram indicates that the relative movement of the connector is largest at the lower flange (fig. 2). The movement of the lower flange is accompanied by deformations of the web. As those deformations are limited to sections of the web located close to the lower flange, the total deflection of the web at the junction with the lower flange must necessarily remain small as long as the stresses in the channel are elastic, or until crushing of the concrete permits the web to deform over a greater length. Thus as long as the steel of the connector does not yield and the

concrete of the slab does not crush, the relative movement between the channel and the slab, and therefore also the slip between the beam and the slab, remain small.

Critical stress

The maximum stress in a channel connector, f_{max}, is located at the junction of the lower flange and the web, and the maximum pressure on the concrete, f_{co}, is located adjacent to the lower flange of the channel. The magnitude of these maximum stresses may be evaluated with the aid of the idealized equivalent pressure distribution shown in figure 2 as a cross-hatched area.

The total pressure on the concrete must be equal to the load Q acting on the connector. It follows from figure 2 that the maximum pressure on the concrete is:

$$f_{co} = \frac{Q}{(h + \frac{1}{2}t)\, w} \qquad (1)$$

If the connector is considered to be a cantilever fixed at its lower end and loaded with the cross-hatched area, the maximum stress in the connector is:

$$f_{max} = \frac{Q}{k_1 (h + \frac{1}{2}t)\, w} \qquad (2)$$

where:

$$k_1 = \frac{1.6}{\frac{f_{co}}{f'_c} + 1.8} \qquad (3)$$

The quantity k_1 is an empirical coefficient evaluated from the tests of channel shear connectors. (Equations 1 and 2 were obtained originally by simplification of more complex theoretical formulas. Ref. *6D*).

It has been explained in the preceding discussion that the slip between the slab and the beam of a composite T-beam will be small as long as the stresses in the channel remain in the elastic range and the concrete does not crush. Thus the limiting stress for the steel is the yield-point stress and the limiting pressure for the concrete is that which causes crushing.

Only the concrete adjacent to the lower flange of the channel is subjected to high pressure. This high pressure is localized on a very small area of the slab so that the concrete at this location is in a state of confined compression. It is known that concrete under such state of stress can resist stresses appreciably greater than its cylinder strength. Furthermore, the magnitude of this pressure depends on the deformation of the channel web. Since the deformations of the web are small until the commencement of yielding of the steel, it seems unlikely that crushing of the concrete would occur before the connector has yielded.

The tests of channel shear connectors have shown conclusively that yielding occurs before crushing of the concrete. In all tests of individual shear connectors, the load causing first yielding of the channel was reached without any visible signs of distress in the concrete (ref. *6B*). Furthermore, the maximum test loads on all specimens were substantially higher than the loads at first yielding of the channels (table 1), and in most specimens no apparent signs of distress in the concrete could be observed in the vicinity of the

Table 1.—Load capacity of channel shear connectors beyond first yielding of connectors [1]

Channel size	Concrete strength	$\frac{Q_{ult}}{Q_{yp}}$
	P.s.i.	
3-in., 4.1-lb.	3,310	2.98
4-in., 5.4-lb.	2,070	2.57
4-in., 5.4-lb.[2]	2,810	2.48
4-in., 5.4-lb.	6,320	2.96
4-in., 5.4-lb.[3]	2,650	2.36
4-in., 7.25-lb.	2,300	2.06
4-in., 7.25-lb.	5,050	3.26
4-in., 7.25-lb.[3]	2,170	1.76
4-in., 13.8-lb.	2,670	2.21
5-in., 6.7-lb.	3,260	2.62

[1] Loads Q_{ult} and Q_{yp} on shear connectors measured in the test of push-out specimens (ref. *6A*).

[2] Connector was 4 inches long; all other connectors were 6 inches long.

[3] Specimen had a concrete fillet between the I-beam and the slab.

region of high compressive stresses even at the maximum loads (ref. *6C*). Thus the critical stress for an individual shear connector is the yield-point stress, and the corresponding limiting load-carrying capacity of a shear connector may be computed from equation 2.

Conclusions

Two major conclusions may be drawn from the investigations which are the basis for the preceding discussions:

1. In a composite steel and concrete T-beam, it is desirable that full composite action be retained at all levels of loading, up to the ultimate load.

2. If the shear connection consists of flexible channel shear connectors, full composite action will be retained as long as the stresses in the connectors do not exceed the yield point.

Any procedure for the design of channel shear connectors based on these two requirements must necessarily be an ultimate design procedure, since it requires an evaluation of the capacity of the connectors corresponding to the ultimate moment-carrying capacity of the T-beam. Nevertheless, since current procedures for the design of I-beam bridges are based on working stresses and not on ultimate capacity, it is desirable to base the design of connectors also on working stresses. This can be done by modifying the ultimate design equations through the use of certain conversion factors, as shown in the following paragraphs.

Load Capacity of Channel Shear Connectors

Capacity at ultimate load on beam

When the load applied to a composite T-beam reaches the full moment capacity of the beam, the maximum steel stress in the connectors should not exceed the yield-point stress:

$$f_{max} = f_{yp} \qquad (4)$$

The load on the shear connectors at this stage may be found from equation 2 by substituting from equations 4, 3, and 1 for f_{max}, k_1, and f_{co}:

$$\frac{Q_{yp}}{(h + \frac{1}{2}t)\, w f'_c} = -0.9 + \sqrt{0.81 + 1.6 f_{yp}/f'_c} \qquad (5)$$

Equation 5 is shown graphically as a solid line in figure 3, in which the quantity on the left side of the equation is plotted against the ratio f_{yp}/f'_c. For structural steel with a specified yield-point stress of 33,000 p.s.i., and for concretes having compressive strengths varying from 2,500 to 5,000 p.s.i., the ratio f_{yp}/f'_c varies from 6.6 to 13.2. Within this range, which represents most practical conditions, equation 5 may be replaced by the simpler expression:

$$\frac{Q_{yp}}{(h + \frac{1}{2}t)\, w f'_c} = \sqrt{f_{yp}/f'_c} \qquad (6)$$

Equation 6 is shown in figure 3 as a broken line. It can be seen that within the practical range of values for f_{yp}/f'_c the maximum difference between the loads given by equations 5 and 6 is less than 5 percent, and for a range of f_{yp}/f'_c from 5.5 to 30 (corresponding to f_{yp} between 33,000 and 60,000 p.s.i., and f'_c between 2,000 and 6,000 p.s.i.) the maximum difference is less than 10 percent. For the common value of $f_{yp}/f'_c = 9.43$ ($f_{yp} = 33{,}000$ p.s.i., $f'_c = 3{,}500$ p.s.i.) the two equations yield practically identical results.

The loads computed from equation 6 are compared with the results of tests of individual shear connectors (ref. *6A*) in figure 4. The computed loads were obtained from equation 6 for $f_{yp} = 33{,}000$ p.s.i. and for the actual concrete cylinder strengths for the particular specimens. The meas-

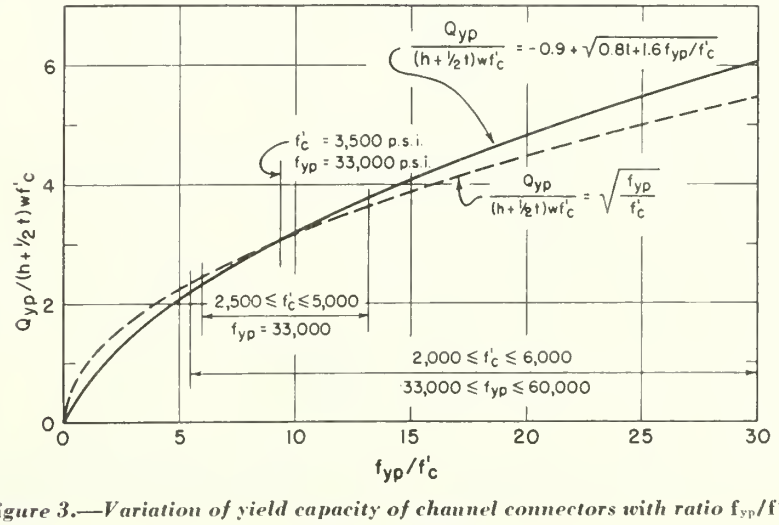


Figure 3.—Variation of yield capacity of channel connectors with ratio f_{yp}/f'_c.

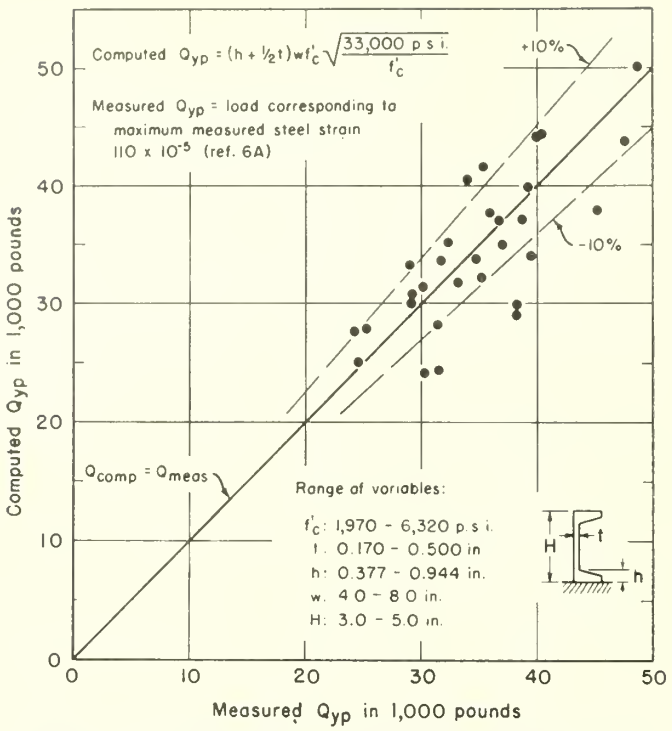


Figure 4.—Comparison of computed and measured yield capacities of channel connectors.

ured loads are those giving a measured maximum strain in the connector web of 110 × 10^{-5}, corresponding to a stress of 33,000 p.s.i. In spite of the wide range of variables included in this figure, all data group well around the line representing $Q_{measured} = Q_{computed}$.

Capacity at design load

If it is assumed that the connection between the beam and the slab of a composite **T**-beam is furnished only by the individual connectors, the load on any connector is the product of the horizontal shear per unit length of the beam and the spacing s of the connectors. As long as the stress distribution in the **T**-beam is elastic, the load on a connector may be computed from the equation:

$$Q = \frac{V A_c y_c}{n I} s \qquad (7)$$

If a portion of the beam has yielded, equation 7 gives approximately correct values of the loads acting on shear connectors located at sections of the **T**-beam at which yielding did not occur, whereas the load is somewhat higher on the connectors located at sections at which the beam has yielded. However, overstressing of a few connectors is not critical from the standpoint of composite action (ref. *6H*). Thus equation 7 may be used for computing the loads on connectors both at working and ultimate loads. This makes it possible to compute the ratio of the loads on a connector at first yielding of the connector, Q_{yp}, to that at the design load, Q_{des}, as the ratio of the corresponding vertical shears:

$$\frac{Q_{yp}}{Q_{des}} = \frac{V'_{ult}}{V_{des}} \qquad (8)$$

If it is desired that the factor of safety against first yielding of shear connectors be at least equal to the factor of safety of the composite **T**-beam against ultimate load, the magnitude of the ratio in equation 8 may be evaluated from the moment capacities of the **T**-beam at ultimate and at design loads. The relation between the shears and moments is different for beams built without temporary supports and for beams built with temporary supports. Therefore these two cases will be treated separately.

Beams without shoring

If a composite **T**-beam is built without any temporary supports, the steel I-beam must carry both its own dead load and the weight of the slab. Thus the shear connectors are required to resist only the live loads, and equation 8 may be written in the following form:

$$\frac{Q_{yp}}{Q_{des}} = \frac{V_{ult} - V_{DL}}{V_{LL}} \qquad (9)$$

It can be shown that the relation between the shear and moments may be written as:

$$\frac{V_{ult} - V_{DL}}{V_{LL}} = \frac{M_{ult} - M_{DL}}{M_{LL}} \qquad (10)$$

By definition:

$$M_{ult} = S_{pl} f'_{yp} \text{ and } M_{LL} = S_c f_s \frac{1}{1 + \frac{S_c M_{DL}}{S_I M_{LL}}}.$$

Also:

$$\frac{M_{DL}}{M_{LL}} = C_M \text{ and } \frac{S_c}{S_I} = C_S.$$

If these expressions are substituted into equations 10 and 9, the ratio of loads on a shear connector may be expressed in the form:

$$\frac{Q_{yp}}{Q_{des}} = \frac{S_{pl}}{S_c} \frac{f'_{yp}}{f_s} (1 + C_S C_M) - C_M \qquad (11)$$

For structural steel for highway bridges, f_s = 18,000 p.s.i. and f'_{yp} = 33,000 p.s.i. The ratio of the plastic to the elastic section modulus, S_{pl}/S_c, depends on the dimensions of the slab and the beam, on the ratio n of the moduli of elasticity of steel and concrete, on the cylinder strength of the concrete, and on the yield-point stress of the steel beams. The magnitude of this ratio has been computed for a wide range of standard wide-flange beams of structural steel combined with concrete slabs of various sizes and strengths.

Representative results of these studies are presented in table 2. It can be seen that for **T**-beams in which the compressive stresses at ultimate are carried by the slab alone, that is, the neutral axis lies above the top flange of the steel beam (case I in fig. 1), the ratio S_{pl}/S_c varies approximately from 1.38 to 1.49. For **T**-beams in which the neutral axis at ultimate intersects the steel section, that is, the steel beam is not used to full tensile capacity because a portion of the compressive forces has to be resisted by the steel beam (case II in fig. 1), the value of the ratio S_{pl}/S_c decreases with decreasing contribution of the slab to the resisting compressive forces. The lower limit for this case is a steel I-beam without any slab, for which the ratio S_{pl}/S_c varies approximately from 1.10 to 1.20, depending on the proportions of the I-beam section. In most practical applications, the relative proportions of the slab

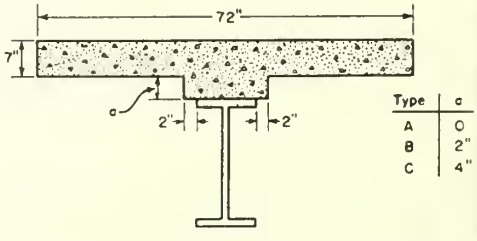


Figure 5.—Cross section (see table 2).

Table 2.—Ratios of plastic section moduli to elastic section moduli for composite T-beams

Beam	S_{pl}/S_c (f'_{yp} = 33,000 p.s.i.)				
	Cross-section type A,[1] f'_c as given below			Type B,[1] f'_c = 3,000 p.s.i.	Type C,[1] f'_c = 3,000 p.s.i.
	2,000 p.s.i.	3,000 p.s.i.	5,000 p.s.i.		
16WF36	1.48	1.45	1.38	1.45	1.42
21WF73	1.43	1.47	1.46	1.49	1.49
30WF116	[2] 1.37	1.42	1.44	1.45	1.47
36WF150	[2] 1.35	1.39	1.42	1.43	1.45
36WF300	[2] 1.26	[2] 1.29	[2] 1.32	[2] 1.30	[2] 1.30

[1] See figure 5 for cross-section dimensions.
[2] At ultimate load a portion of the compressive forces is resisted by the steel I-beam (fig. 1, case II); in all other cases all compression is resisted by the slab (fig. 1, case I).

and the I-beam will be such that at ultimate load the neutral axis will be located in the slab. Thus the ratio S_{pl}/S_c will vary within relatively narrow limits and may be replaced in equation 11 by a value of 1.45 without introducing any serious error.

If the shear connectors are made of structural steel, that is $f_{yp} = 33{,}000$ p.s.i. and if, in addition, $f'_{yp} = 33{,}000$ p.s.i. and $f_s = 18{,}000$ p.s.i., and if Q_{yp} is substituted from equation 6, then equation 11 may be written in the following form:

$$\frac{Q_{des}}{(h + \tfrac{1}{2}\,t)\ w\,f'_c} = \frac{1}{0.8 + (0.8\,C_S - 0.3)\ C_M}\sqrt{\frac{3000\text{ p.s.i.}}{f'_c}} \quad (12)$$

Equation 12 is a design formula for channel shear connectors for beams built without shores. For any particular composite **T**-beam the quantities C_M, C_S, and f'_c are assumed to be known in advance of the design of shear connectors, and the right-hand side of equation 12 is therefore known. To simplify the use of equation 12, values of $(h + \tfrac{1}{2}\,t)$ for the steel channels now being rolled are listed in table 3. Because of the assumptions made in its derivation, and because of the limits of the experimental evidence on which it is based, equation 12 is applicable only to flexible channel shear connectors made of rolled channels of structural steel attached to rolled wide-flange I-beams, also of structural steel. It is applicable only to channels at least 3 inches deep and having a length w not less than six times the maximum thickness of the channel flange h (ref. *6E*).

Beams with shoring

If a composite **T**-beam is built with temporary supports, both the dead load and the live load are carried by the composite section and the shear connectors must resist both dead and live load. Equation 8 may then be written as:

$$\frac{Q_{yp}}{Q_{des}} = \frac{V_{ult}}{V_{LL} + V_{DL}} \quad (13)$$

The relation between the shear and the moments may be expressed as:

$$\frac{V_{ult}}{V_{LL} + V_{DL}} = \frac{\dfrac{M_{ult}}{M_{LL} + M_{DL}}(1 + C_M) - C_M + C_V}{1 + C_V} \quad (14)$$

where:

$$C_M = \frac{M_{DL}}{M_{LL}} \text{ and } C_V = \frac{V_{DL}}{V_{LL}}.$$

By definition, $M_{ult} = S_{pl}f'_{yp}$ and $(M_{LL} + M_{DL}) = S_c f_s$. Thus equation 13 may be written as:

$$\frac{Q_{yp}}{Q_{des}} = \frac{\dfrac{S_{pl}}{S_c}\dfrac{f'_{yp}}{f_s}(1 + C_M) - C_M + C_V}{1 + C_V} \quad (15)$$

and for beams and shear connectors made of structural grade steel the equation for the design load is:

$$\frac{Q_{des}}{(h + \tfrac{1}{2}\,t)\ wf'_c} = \frac{1 + C_V}{0.8 + 0.5\,C_M + 0.3\,C_V}\sqrt{\frac{3{,}000\text{ p.s.i.}}{f'_c}} \quad (16)$$

For any particular composite **T**-beam, C_M is a constant but C_V varies from point to point along the span. Thus the use of equation 16 would result in a rather cumbersome design. However, the design procedure would become much more simple if the variable parameter C_V could be replaced by a constant. It can be shown that for $C_V = 0$, which represents the lowest possible value of C_V, equation 16 yields the minimum values of Q_{des}. Accordingly, the following equation will yield a conservative value of Q_{des} for composite **T**-beams built with shoring:

$$\frac{Q_{des}}{(h + \tfrac{1}{2}\,t)\ w\,f'_c} = \frac{2}{1.6 + C_M}\sqrt{\frac{3{,}000\text{ p.s.i.}}{f'_c}} \quad (17)$$

Equation 17 may be obtained from equation 12 by setting $C_S = 1$.

Repeated Loading

The design equations 12 and 17 are based on tests and on strengths and allowable stresses relating only to short-time static loading. However, shear connectors in a bridge are subjected to repeated loading, and connectors located close to midspan are subjected to full reversal of stress for each passage of a vehicle. It is important, therefore, to determine whether a design based on equations 12 and 17 will be safe also from the standpoint of repeated loading.

The critical stresses at the design load may be computed from equations 12 or 17, and equations 1, 2, and 3. The absolute upper limit for the maximum stress in the concrete ($C_M = 0$, $C_S = 1$) may be expressed as:

$$\frac{f_{co}}{f'_c} = 1.25\sqrt{\frac{3{,}000\text{ p.s.i.}}{f'_c}} \quad (18)$$

and the upper limit for the maximum stress in the steel may be expressed as:

$$f_{max} = \frac{1.25\,f'_c}{k_1}\sqrt{\frac{3{,}000\text{ p.s.i.}}{f'_c}} \quad (19)$$

Since both of these critical stresses are functions solely of the cylinder strength of the concrete, their magnitude may easily be evaluated for the practical range of variables. This was done and the results are compared with safe values for repeated loading in figure 6.

In figure 6A the values computed from equation 18 are compared with the results of repeated load tests of small-scale composite **T**-beams (ref. *4B*). In the majority of these tests the dimensions of the channels were such that equation 1 for the maximum pressure on the concrete was applicable. Thus the pressure on the concrete could be evaluated and the fatigue strength of the connectors determined in terms of f_{co}/f'_c. This has been done for specimens failing at 600,000 repetitions of load or less (ref. *5*) and the limiting curve obtained from these studies is included in figure 6A. It should be mentioned, however, that with the possible exception of concrete strengths in the neighborhood of 2,000 p.s.i., the fatigue failure occurred in the channels and not in the concrete. In other words, the fatigue strength of the concrete adjacent to the channel was even higher than that indicated in figure 6A.

The fatigue strength of the channel shear connectors in terms of the maximum steel stress could not be determined from the small-scale experiments, since the dimensions of these specimens were such that equation 2 was not strictly

Table 3.—Relative efficiency of channel shear connectors

Channel type and size	Dimensions[1]			Relative efficiency of channels[2]
	$h + \tfrac{1}{2}\,t$	h	h'	
	In.	*In.*	*In.*	*Percent*
AMERICAN STANDARD:				
3-inch:				
4.1-lb.	0.462	0.377	0.170	100
5.9-lb.	.506	.377	.170	90
6.0-lb.	.555	.377	.170	82
4-inch:				
5.4-lb.	.503	.413	.180	82
7.25-lb.	.573	.413	.180	70
5-inch:				
6.7-lb.	.545	.450	.190	72
9.0-lb.	.613	.450	.190	62
6-inch:				
8.2-lb.	.587	.487	.200	63
10.5-lb.	.644	.487	.200	54
13.0-lb.	.706	.487	.200	48
7-inch:				
9.8-lb.	.628	.523	.210	57
12.25-lb.	.680	.523	.210	48
14.75-lb.	.733	.523	.210	44
CAR BUILDING:				
3-inch:				
7.1-lb.	.546	.390	.313	68
9.0-lb.	.640	.390	.313	63
4-inch:				
13.8-lb.	.781	.531	.469	50
SHIP BUILDING:				
6-inch:				
12.0-lb.	.570	.413	.337	42
15.1-lb.	.678	.521	.429	38
15.3-lb.	.610	.440	.330	35
16.3-lb.	.708	.521	.429	38
18.0-lb.	.717	.530	.420	35
7-inch:				
17.6-lb	.708	.521	.429	36
19.1-lb.	.729	.554	.446	34
22.7-lb.	.804	.554	.446	31

[1] Dimensions h, h', and t are shown in figure 2.
[2] Expressed as a percentage of the capacity of the 3-in., 4.1-lb. channel.

applicable. Therefore, the values computed from equation 19 are compared in figure 6B with the allowable steel stress for full reversal of loading (ref. *1*).

It can be concluded from the comparisons shown in figure 6 that connectors designed according to equations 12 or 17 will be safe from the standpoint of repeated loading.

Spacing and Cover

Spacing

The design load for a shear connector computed from equation 7 should not exceed the capacity of the connector given by equations 12 or 17. Thus, for a known size of channel, the required spacing s may be evaluated from equations 7 and 12 or 17. Since the maximum vertical design shear V_{des} varies along the lengths of the beam, and since it would seldom be advisable to vary the size of the connectors, the spacing of the connectors obtained by this procedure will vary along the beam.

A shear connector is required not only to transfer the horizontal shear from the slab to the beam but also to tie the slab and the beam together in the vertical direction. If the spacing of the connectors is not excessive, the upper flange of the channel provides an effective tie-down. The tests of full-size composite T-beams indicated that a spacing of 18 inches, equal to three times the thickness of the slab, was satisfactory, whereas a spacing of 36 inches, equal to six times the thickness of the slab, was unsatisfactory from the standpoint of vertical tie-down (ref. *6J*). It is recommended that the maximum spacing of channel shear connectors be not greater than four times the thickness of the slab, but in no case greater than 24 inches.

Cover

The concrete adjacent to the channel flange connected to the beam is subjected to localized high compressive stresses which can be sustained only if sufficient restraint is offered by the flange of the beam and by the surrounding concrete subjected to smaller stresses. It is important, therefore, that a sufficient cover of concrete be provided at the ends of the channel. It is believed that a 2-inch cover will offer the necessary restraint. (Some test data relating to this question may be found in ref. *6C*). For the same reason, the connector should be shorter than the width of the flange of the I-beam, and it is recommended that the connector should not extend closer than 1 inch to the edge of the beam flange —that is, the length of the connector should be at least 2 inches less than the width of the flange. Furthermore, if the slab is built with a concrete fillet between the I-beam and the main body of the slab, the width of this fillet should be at least 4 inches more than the length of the connector.

If the slab is built with a concrete fillet between the I-beam and the main body of the slab, shearing stresses of considerable magnitude exist in this fillet. It is desirable, therefore, that the connectors extend some distance into the main body of the slab. It is recommended that this distance be at least 2 inches, and preferably 3 inches.

Design of Connecting Welds

The most common method of connecting a shear connector to the top flange of an I-beam is by welding. In all tests made at the University of Illinois, the connection was made by continuous welds extending the full width of the channel at both the heel and toe.

Static loading

Tests of individual shear connectors indicated that the ultimate strength of the welded connection may be estimated as the product of the throat area of the welds and 80,000 p.s.i. (ref. *6C*). The allowable stress specified by the current specifications (ref. *2*) is 12,400 p.s.i. Thus the factor of safety against failure of welds designed for the allowable stress and for the full load capacity of the connector is 6.4 at working loads, and at least 2.4 at first yielding of the connectors. Since the ratio of the ultimate strength to the yield strength of the connector itself varies approximately from 2.0 to 3.0 (table 1), the corresponding minimum value of 2.4 for welds seems to be reasonable. Thus, the effective area of the welds for shear connectors may be computed as:

$$A_w \geq \frac{Q_{des}}{12{,}400 \text{ p.s.i.}} \qquad (20)$$

Repeated loading

Current specifications (ref. *2*) require that the effective area of welds subject to repeated loading be:

$$A_w \geq \frac{\text{Max. } Q - \frac{1}{2} \text{ Min. } Q}{10{,}000 \text{ p.s.i.}} \qquad (21)$$

In this formula the terms "Max." and "Min." refer to the maximum and minimum loads on shear connectors. "Max." refers to the numerically greater load and should always be taken as a plus quantity. If the minimum load has opposite direction to that of the maximum load, the "Min." should be taken as a minus quantity. The magnitudes of the Max. Q and Min. Q should be computed from equation 7; in this equation V should be the maximum or minimum shear caused by that portion of the design loads acting on the bridge which contributes to the load acting on shear connectors, and s the actual spacing of the shear connectors.

Size of welds

The maximum allowable size of welds is governed by the dimensions of the channel. The weld on the front of the channel cannot be higher than the minimum thickness of the channel flange, h' (fig. 2). The height of the weld on the back of the channel is also limited since it is undesirable for the weld to extend to the section of maximum steel stresses in the channel. It is recommended, therefore, that the weld on the back of the channel be always at least one-eighth inch lower than h. Values of h and h' for various channels are listed in table 3.

Relative Efficiency of Channels of Various Sizes

The load-carrying capacity of a channel shear connector is proportional to the sum of the maximum flange thickness h and one-half of the web thickness t (equations 12 and 17). Thus the contribution of the web to the load-carrying capacity of the connector is relatively small. Furthermore, the capacity is independent of the depth of the channel. Since the value of h for rolled channels varies only from 0.377 inch for the lightest to 0.554 inch for the heaviest types suitable for shear connectors, it may be expected that from the standpoint of steel economy alone the lightest channels will be the most efficient. This point is illustrated in table 3.

In the last column of table 3, termed "Relative efficiency of channels," the relative load-carrying capacity per pound of steel in the channel is listed for various channels as a percentage of the capacity of the lightest channel. It is evident from this table that the efficiency generally decreases with increasing weight, whereas for any particular depth of channel the lightest channel is the most efficient.

The load-carrying capacity of a channel connector increases also with increasing strength of concrete. In practical design, however, the strength of concrete is governed by other considerations and for the design of connectors is a fixed quantity. Thus the designer will generally

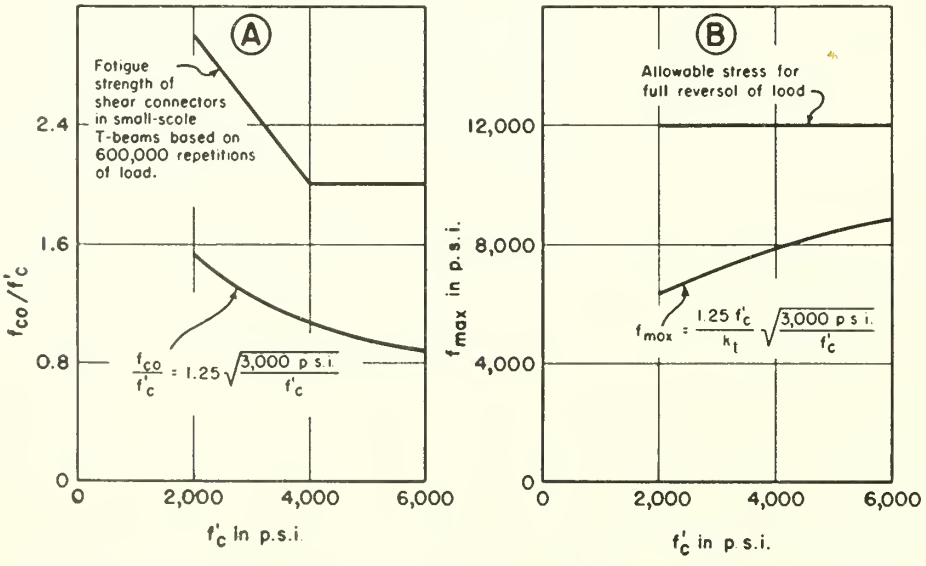


Figure 6.—Variation of f_{co}/f'_c and f_{max} with concrete strength at design load.

be able to vary the capacity of the connector only by varying the dimensions and the spacing of the connectors.

Concluding Remarks

The procedure for the design of channel shear connectors presented in this article is based on the assumption that all of the horizontal shear at the junction between the concrete slab and the steel I-beams is transferred through the shear connectors. At working loads this assumption may be overly conservative since bond may exist between the two elements. If bond is present initially, there is some evidence from both field and laboratory that the presence of strong shear connectors can prevent the destruction of this bond under service conditions. It is unlikely, however, that the bond would survive any serious overstress of the structure, and once the bond is broken it cannot be restored. It is therefore the opinion of the authors that the mechanical shear connectors should be designed for full horizontal shear.

The tests of small-scale T-beams have demonstrated that the presence of bond increases substantially the fatigue life of the shear connectors (ref. *4C*). All specimens tested in fatigue were built with shear connectors, and in most specimens the bond between the steel beam and the concrete slab was broken before the fatigue test was begun, with the result that all of the horizontal shear was carried by the shear connectors. In a few specimens, however, the bond failed only during the fatigue tests; and in two specimens the bond remained unbroken throughout the tests. These specimens with bond withstood more repetitions of load than the corresponding specimens without bond, and the two specimens in which the bond did not break at all withstood over 2,000,000 repetitions of load without any signs of damage. Since it is most probable that bond exists in actual structures, at least for some time, it might be possible to relax the design requirements based on repeated loading. It can be seen from the previous paragraphs that such a provision would affect only the design of welds.

The analysis on which the design procedure is based contains empirical parameters determined from the tests of channel shear connectors. Consequently, the use of this procedure must be limited to channel shear connectors within the range of variables included in the tests. Fortunately, the range of variables in the tests was great enough to accommodate all channels deeper than 3 inches. It is possible that the procedure might be applicable also to some other rolled steel shapes. However, any such extension of the applicability of the procedure would have to be substantiated by tests.

The procedure for the design of channel shear connectors and its limitations may be summarized as follows:

1. For composite beams built without temporary supports, flexible channel shear connectors made of structural steel and attached to wide-flange beams made of structural steel may be designed according to equation 12.
2. For composite beams built with temporary supports, channels made of structural steel attached to wide-flange beams made of structural steel may be designed according to equation 17.
3. The spacing of the connectors should be determined from equation 7, and should not be greater than four times the slab thickness, or 24 inches.
4. The length of channel connectors should not be greater than the width of the supporting flange of the I-beam minus 2 inches.
5. If the slab is built with a concrete fillet between the I-beam and the main body of the slab, the width of the fillet should be at least 4 inches more than the length of the connectors. The connectors should extend at least 2 inches above the bottom of the main body of the slab.

REFERENCES

(*1*) American Association of State Highway Officials
Standard specifications for highway bridges. American Association of State Highway Officials, Washington, D. C., 1949, art. 3.6.5.

(*2*) American Welding Society
Standard specifications for welded highway and railway bridges. American Welding Society, New York, 1947.

(*3*) Siess, C. P.
Composite construction for I-beam bridges. Transactions of the American Society of Civil Engineers, New York, vol. 114, 1949, pp. 1023-1045.

(*4*) Siess, C. P., Viest, I. M., and Newmark, N. M.
Studies of slab and beam highway bridges, part III: Small-scale tests of shear connectors and composite T-beams. University of Illinois Engineering Experiment Station Bulletin No. 396, Urbana, 1952.

(*4A*) *Bulletin 396,* section 25, pp. 63-71.

(*4B*) *Bulletin 396,* chapter V, pp. 81-109.

(*4C*) *Bulletin 396,* section 35, pp. 91-95.

(*5*) Viest, I. M.
Full-scale tests of channel shear connectors and composite T-beams. A Ph. D. thesis, University of Illinois, Urbana, 1951, section 44, pp. 113-116.

(*6*) Viest, I. M., Siess, C. P., Appleton, J. H., and Newmark, N. M.
Studies of slab and beam highway bridges, part IV: Full-scale tests of channel shear connectors and composite T-beams. University of Illinois Engineering Experiment Station Bulletin No. 405, Urbana, 1952.

(*6A*) *Bulletin 405,* table 5, pp. 26-27.

(*6B*) *Bulletin 405,* section 12, pp. 28-45.

(*6C*) *Bulletin 405,* section 15, pp. 53-55.

(*6D*) *Bulletin 405,* section 27, pp. 74-76.

(*6E*) *Bulletin 405,* section 28, pp. 76-78.

(*6F*) *Bulletin 405,* sections 38-41, pp. 98-118.

(*6G*) *Bulletin 405,* table 24, p. 116.

(*6H*) *Bulletin 405,* section 42, pp. 118-123.

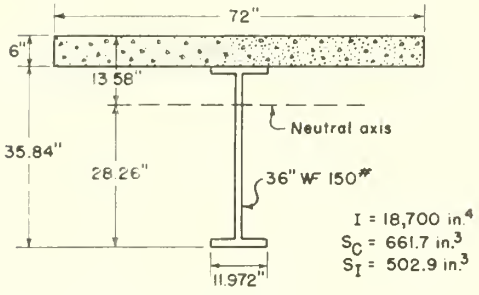
(*6J*) *Bulletin 405,* section 44, pp. 124-126.

Basic information

Span of beams: 60 ft.
Beam spacing: 6 ft.
For effective slab width, see A.A.S.H.O. Specifications, 1949, Art. 3.9.2.
Structural grade steel.
Concrete strength: 3,000 p.s.i.
Beams not shored during construction.
Loading: H20-S16-44.
Allowable stresses (A.A.S.H.O. Specifications, 1949, revised 1950):

Steel: $f_s = 18{,}000$ p.s.i.
Concrete: $f_c = 1.200$ p.s.i.
$n = 10$.

Cross-section of T-beam:



Design of shear connectors

Horizontal shear (equation 7):

$$\frac{Q_{des}}{s} = \frac{V\ 72 \times 6\ (13.58 - 3.00)}{10 \times 18{,}700} = 0.0244\ V.$$

Vertical shear at end:

$$V = (16{,}000 + \frac{6.0}{5.5}\,16{,}000\,\frac{60-14}{60} + \frac{6.0}{5.5}\,4{,}000\,\frac{60-28}{60})\,(1 + \frac{50}{60+125}) = 40{,}300 \text{ lb.}$$

Vertical shear 12 ft. from end:

$$V = \frac{6.0}{5.5}\,(16{,}000\,\frac{60-12}{60} + 16{,}000\,\frac{60-12-14}{60} + 4.000\,\frac{60-12-2\times 14}{60})\,(1 + \frac{50}{60-12+125}) = 32{,}600 \text{ lb.}$$

Vertical shear 24 ft. from end:

$$V = \frac{6.0}{5.5}\,(16{,}000\,\frac{60-24}{60} + 16{,}000\,\frac{60-24-14}{60} + 4{,}000\,\frac{60-24-2\times 14}{60})\times 1.3 = 22{,}700 \text{ lb.}$$

Use 3-in., 4.1-lb channel of maximum allowable length $w = 11.97 - 2 \doteq 10$ in.
From table 3: $h + \frac{1}{2}t = 0.462$.
From the design of beams:

$$C_M = \frac{M_{DL}}{M_{LL}} = 0.483;\ C_S = \frac{S_C}{S_I} = 1.316.$$

From equation 12:

$$Q_{des} = \frac{0.462 \times 10 \times 3{,}000}{0.8 + (0.8 \times 1.316 - 0.3)\,0.483}\sqrt{\frac{3{,}000}{3{,}000}} = 11{,}910 \text{ lb.}$$

Spacing at end:

$$s = \frac{11{,}910}{0.0244 \times 40{,}300} = 12.1 \text{ in.}$$

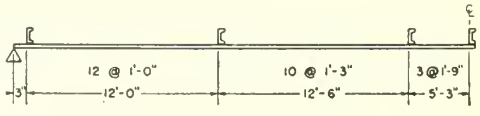
Spacing 12 ft. from end:

$$s = \frac{11{,}910}{0.0244 \times 32{,}600} = 15.0 \text{ in.}$$

Spacing 24 ft. from end:

$$s = \frac{11{,}910}{0.0244 \times 22{,}700} = 21.6 \text{ in.}$$

Spacing diagram:



Connecting welds

Weld area required at end:

$V_{max} = 40{,}300$ lb.; $V_{min} = 0$.

From equation 7: $Q_{max} = 0.0244 \times 40{,}300 \times 12 = 11{,}800$ lb.; $Q_{min} = 0$.

$$A \geqslant \frac{11{,}800}{10{,}000} = 1.18 \text{ in.}^2$$

Weld area required 12 ft. from end:

$V_{max} = 32{,}600$ lb.;

$$V_{min} = \frac{6.0}{5.5}\,(16{,}000\,\frac{60-12}{60} - 16{,}000) \times 1.3 = -4{,}540 \text{ lb.}$$

$Q_{max} = 0.0244 \times 32{,}600 \times 15 = 11{,}930$ lb.
$Q_{min} = -0.0244 \times 4{,}540 \times 15 = -1{,}660$ lb.

$$A \geqslant \frac{11{,}930 + \frac{1}{2}\,1{,}660}{10{,}000} = 1.28 \text{ in.}^2$$

Weld area required 24 ft. from end:

$V_{max} = 22{,}700$ lb.

$$V_{min} = \frac{6.0}{5.5}\,(16{,}000\,\frac{60-24}{60} + 16{,}000\,\frac{60-24+14}{60} - 2 \times 16{,}000) \times 1.3 = -12{,}860 \text{ lb.}$$

$Q_{max} = 0.0244 \times 22{,}700 \times 21 = 11{,}630$ lb.
$Q_{min} = -0.0244 \times 12{,}860 \times 21 = -6{,}590$ lb.

$$A \geqslant \frac{11{,}630 + \frac{1}{2}\,6{,}590}{10{,}000} = 1.49 \text{ in.}^2$$

Weld area required at midspan:

$$V_{max} = -V_{min} = \frac{6.0}{5.5}\,(16{,}000\,\frac{60-30}{60} + 16{,}000\,\frac{60-30-14}{60} + 4{,}000\,\frac{60-30-2\times 14}{60}) \times 1.3 = 17{,}590 \text{ lb.}$$

$$Q_{max} = -Q_{min} = 0.0244 \times 17{,}590 \times 21 = 9{,}010 \text{ lb.}$$

$$A \geqslant \frac{9{,}010 + \frac{1}{2}\,9{,}010}{10{,}000} = 1.35 \text{ in.}^2$$

Area available with $\frac{3}{16}$-in. weld at the heel and toe of channel,

$A = 0.707 \times \frac{3}{16} \times 2 \times 10 = 2.65$ in.^{2}, is sufficient.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1954

FOR OFFICIAL USE

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPRVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$11,654	$12,717	$6,766	179.7	$8,373	$4,210	68.2	$45,021	$22,935	495.8	$66,111	$33,911	743.7
Arizona	3,362	7,959	5,643	177.9	1,009	645	9.0	5,363	3,610	68.5	14,331	10,096	255.4
Arkansas	9,399	8,046	4,722	316.7	7,100	3,543	161.6	10,600	5,334	246.1	25,746	13,599	724.4
California	16,596	24,031	12,462	152.4	13,377	6,918	31.2	86,540	41,499	203.0	123,948	60,879	386.6
Colorado	10,728	2,432	1,357	39.7	1,943	1,090	59.5	19,146	10,368	190.4	23,521	12,815	289.6
Connecticut	12,036	450	275	2.0	1,047	516	1.0	8,461	4,220	26.0	9,958	5,011	29.0
Delaware	3,205	3,756	1,905	23.1	1,404	699	11.9	3,603	1,797	18.8	8,763	4,401	53.8
Florida	12,168	12,748	6,418	162.1	4,813	2,546	91.5	20,533	10,741	307.7	38,094	19,705	561.3
Georgia	15,919	16,888	8,503	311.6	4,381	2,156	81.9	36,254	17,146	520.9	57,523	27,805	914.4
Idaho	2,916	12,705	8,025	270.2	3,091	1,886	42.8	10,242	6,372	172.2	26,038	16,283	485.2
Illinois	24,158	36,225	18,808	343.6	19,645	9,706	84.0	54,566	29,055	299.7	110,436	57,569	727.3
Indiana	12,416	48,681	25,714	240.1	13,897	6,967	92.2	16,413	8,718	54.8	78,991	41,399	387.1
Iowa	11,672	14,153	7,532	513.6	7,231	3,771	181.7	13,657	7,637	454.6	35,041	18,940	1,149.9
Kansas	10,461	14,546	7,270	938.5	4,544	2,368	259.2	12,947	6,320	665.0	32,037	15,958	1,862.7
Kentucky	10,510	9,351	5,017	108.9	7,104	3,555	91.4	21,136	11,084	203.7	37,591	19,656	404.0
Louisiana	8,407	21,635	10,063	155.8	5,102	2,546	43.1	25,483	12,238	113.8	52,220	24,947	312.7
Maine	5,254	4,155	2,149	30.9	1,164	563	4.9	11,197	5,611	84.7	16,516	8,323	120.5
Maryland	8,358	13,435	7,083	46.2	2,869	1,501	19.4	9,866	5,406	33.3	26,170	13,990	98.9
Massachusetts	8,483	2,897	1,583	6.1	13,231	6,566	13.3	42,346	19,679	28.2	58,474	27,828	47.6
Michigan	19,443	26,169	13,154	415.5	6,522	3,724	126.5	36,217	16,861	200.4	69,308	33,739	742.4
Minnesota	15,002	9,410	5,203	804.9	6,186	3,196	152.1	12,789	6,760	201.8	28,385	15,159	1,158.8
Mississippi	7,910	12,442	6,337	332.8	4,988	2,464	161.9	19,536	9,862	448.2	36,966	18,663	942.9
Missouri	15,915	18,032	9,385	814.4	7,560	4,058	184.6	47,141	23,134	365.6	73,133	36,577	1,364.6
Montana	6,570	20,298	12,310	603.2	4,227	2,530	98.1	17,731	10,703	260.2	42,256	25,543	961.5
Nebraska	14,796	18,882	9,888	647.7	9,733	5,082	212.9	7,426	4,304	211.0	36,041	19,274	1,071.6
Nevada	7,935	2,727	2,283	96.0	2,441	2,027	21.5	6,015	4,993	111.8	11,183	9,303	229.3
New Hampshire	3,108	5,174	2,587	34.7	1,226	603	2.9	5,153	2,667	31.3	11,553	5,857	68.9
New Jersey	10,535	13,694	4,322	51.9	4,865	2,323	9.7	24,425	11,776	17.8	42,984	18,421	79.4
New Mexico	5,754	4,110	2,552	133.7	2,412	1,528	51.3	10,072	6,291	224.2	16,594	10,371	409.2
New York	38,818	71,118	37,296	257.8	45,386	22,144	42.2	136,794	64,086	343.4	253,298	123,526	643.4
North Carolina	13,668	19,761	9,835	483.5	5,105	2,264	98.4	32,865	15,722	394.9	57,731	27,821	976.8
North Dakota	3,342	12,062	6,047	1,228.5	4,738	2,369	288.7	6,501	3,403	336.9	23,301	11,819	1,854.1
Ohio	23,784	8,966	4,918	56.8	19,047	8,001	65.6	78,078	37,245	94.0	106,091	50,164	216.4
Oklahoma	15,009	11,928	6,621	135.1	7,498	3,959	139.8	13,635	7,433	177.4	33,061	18,013	452.3
Oregon	5,627	4,266	2,548	36.6	3,720	2,035	43.0	10,576	6,457	148.0	18,562	11,040	227.6
Pennsylvania	13,220	43,304	20,358	82.3	11,209	5,069	32.7	95,291	46,577	177.8	149,804	72,004	292.8
Rhode Island	4,684	2,952	1,476	40.2	307	153		9,847	4,921	24.6	13,106	6,550	64.8
South Carolina	6,943	12,707	6,990	366.3	2,555	1,287	141.1	15,090	7,461	351.8	30,352	15,738	859.2
South Dakota	2,096	15,445	8,854	815.6	1,817	1,019	97.0	6,967	3,891	377.1	24,229	13,764	1,289.7
Tennessee	12,029	12,336	6,146	500.9	9,305	4,664	260.2	30,631	13,965	250.4	52,272	24,775	1,011.5
Texas	30,721	11,061	5,806	206.9	11,732	6,033	235.7	54,981	29,601	1,024.0	77,774	41,440	1,466.6
Utah	953	8,416	6,363	171.0	1,384	1,038	25.6	8,842	6,783	128.6	18,642	14,184	325.2
Vermont	2,153	3,883	1,942	44.0	830	415	9.2	6,519	3,303	35.7	11,232	5,660	88.9
Virginia	9,479	8,253	3,898	78.7	6,811	3,054	103.8	30,291	14,543	194.6	45,355	21,495	377.1
Washington	5,349	17,028	8,514	242.9	1,925	1,025	27.5	16,839	8,917	106.0	35,792	18,456	376.4
West Virginia	9,043	7,101	3,599	48.5	5,332	2,677	12.3	10,149	5,038	45.7	22,582	11,314	106.5
Wisconsin	6,965	24,054	12,431	313.8	2,525	1,331	85.0	23,130	11,546	192.2	49,709	25,308	591.0
Wyoming	1,708	6,846	4,447	216.7	934	605	22.1	7,717	4,842	137.4	15,497	9,894	376.2
Hawaii	3,211	2,111	1,026	3.2	1,858	924	3.5	9,810	4,677	14.3	13,779	6,627	21.0
District of Columbia	4,160	7,368	3,444	5.5	153	71		12,604	5,814	3.0	20,125	9,329	8.5
Puerto Rico	7,092	10,339	4,762	50.3	652	315	4.0	14,727	6,865	39.9	25,718	11,942	94.2
TOTAL	524,724	719,053	376,637	13,339.0	317,108	159,739	4,106.7	1,271,763	640,511	10,857.2	2,307,924	1,176,887	28,302.9

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

Contents of this publication may be reprinted. Mention of source is requested.

Estimate of User Taxes Paid by Vehicles in Different Type and Weight Groups

BY THE DIVISION OF RESEARCH
BUREAU OF PUBLIC ROADS

Reported by EDWIN M. COPE, Chief,
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Studies Section

In this article an estimate is made of the amounts of State highway-user taxes paid by vehicles of different types and general size groups. Of the total of $3,088 million of State motor-vehicle tax payments made in 1952, fuel-tax payments accounted for $1,968 million or 64 percent; registration-fee payments, $910 million or 29 percent; motor-carrier tax contributions, $64 million or 2 percent; and drivers' licenses, miscellaneous fees, etc., $146 million or 5 percent.

Comparisons established in this study show that passenger cars represented 83 percent of all motor vehicles registered, accounted for 81 percent of the traffic on our highways, and contributed 65 percent of total State road-user tax payments. If panel, pickup, and other light trucks are combined with passenger cars the percentages become 93, 89, and 74, respectively. Medium and heavy trucks and combinations accounted for 6 percent of the registrations, 10 percent of the traffic, and contributed 24 percent of the road-user payments. Tractor-semitrailers and truck-trailers included in the preceding group accounted for 1 percent of the registrations, 3 percent of the travel, and 12 percent of user-tax payments. Buses accounted for less than 1 percent of the registrations and travel, and 2 percent of the user-tax payments.

On the basis of highway-user tax payments per mile of travel, passenger cars and light trucks paid one-half cent per mile, buses paid 1.6 cents, and medium and heavy trucks and combinations paid 1.5 cents. The rate for truck combinations alone is slightly more than 2 cents per mile of travel, tractor-semitrailers paying 2.1 cents and truck-trailers, 2.7.

IN a previous article in PUBLIC ROADS[1] there was presented a comparison of the taxes imposed in different States on a selected group of vehicles. The sole purpose of that study was to compare the rates of taxes of the States. An entirely different, though related, matter is the total highway-user tax payments by the different major groups of vehicles. Information on this subject is of considerable importance to highway authorities, legislatures, and vehicle operators in determining the equitability of the total highway tax burden on various groups of vehicles and in weighing the tax burden on the group against the costs of providing highway service, and the benefits derived from the service.

The work presented here constitutes a series of estimates, and some may disagree with the methods used or the findings reached. Given better basic data, or more time for intensive study of individual phases of the estimates, modification would probably be necessary. It is believed, however, that the results are sufficiently within the areas of reasonableness and general validity to be useful.

[1] *Road user and property taxes on selected motor vehicles, 1953*, by E. M. Cope and R. W. Meadows. PUBLIC ROADS, Vol. 27, No. 7, April 1953.

Although the principal value of this study lies in the findings, an outline of the data on which the study is based, together with a brief review of some of the problems encountered and the assumptions that were made, should be useful to those who may have occasion to evaluate or apply the findings.

In 1952 the States collected a net total of $1,967,831,000 in motor-fuel taxes and related fees. The total registration fees and associated revenues amounted to $1,069,439,000 but, for practical purposes, the $12,859,000 of fines and penalties received have been eliminated, leaving a remainder of $1,056,580,000. This was done on the theory that fines and penalties are not actually road-user revenues even though they are miscellaneous receipts of the highway departments in some States. State motor-carrier taxes collected during the year amounted to $64,036,000. The total of the State road-user taxes considered in this study is, therefore, $3,088,477,000.

Thus precise information is available on the amounts of State registration fees that were paid by automobiles, by trucks, and by buses. Various related fees such as drivers' and chauffeurs' licenses, title fees, etc., can be allocated to various classes of vehicles without fear of substantial error. Motor-carrier taxes can also be allocated with some degree of confidence: their payment is accounted for, primarily, by buses and heavy trucks.

At first glance, it might seem that the allocation of gasoline tax payments to the various groups of vehicles should be fairly easy; but this is not the case. To assign gasoline tax payments to the various groups of vehicles requires the determination of the amounts of travel of each group of vehicles, and this is particularly important among the groups of trucks since different rates of fuel consumption are assigned to each group. The formulation of an acceptable fuel consumption curve is in itself no small task, and relatively minor changes in the rates of fuel consumption assigned would make very substantial changes in the computed tax payments. The yield from fuel taxes accounts for approximately two-thirds of all road-user tax payments. According to the results of this study, motor-fuel taxes constitute 68.1 percent of the total State road-user taxes on automobiles, 63.7 percent of the taxes on buses, and 56.1 percent of the taxes on trucks.

Wherever reference is made in this study to State motor-fuel tax receipts, motor-fuel usage, highway use of special fuels, and State motor-vehicle receipts, the data are taken from the Bureau's published bulletin Highway Statistics 1952. Such information is given therein in tables G–1, G–21, G–25, and MV–2, respectively.

Findings of the Study

The findings of this study can best be described by referring to the summary data given in tables 1 and 2 and in figures 1 and 2, which portray the results graphically. The summary data compare the numbers of vehicles in each visual classification, the user taxes paid, vehicle-miles traveled, average payments per vehicle, and average payments per mile of travel.

Table 1 brings together the classified estimates of tax payments that are described individually in subsequent sections of this article. It will be observed that fuel-tax payments accounted for $1,968 million, or 63.7

Table 1.—Estimate of State highway-user taxes paid in 1952 by major groups of vehicles

Vehicle type	Registration fees	Motor-carrier taxes	Operators' and chauffeurs' license fees	Miscellaneous fees	Motor-fuel taxes	Total	
						Amount	Distribution
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*Percent*
Passenger cars	515,750	------	47,235	71,014	1,353,280	1,987,279	64.34
Buses	13,171	7,268	522	289	37,337	58,587	1.90
Motorcycles	1,769	------	102	407	4,488	6,766	.22
Camp and other light trailers	14,117	------	------	------	----------	14,117	.46
Single-unit trucks:							
Panel and pickup	83,804	------	4,436	5,966	123,156	217,362	7.04
Other 2-axle, 4-tire	18,729	------	836	1,186	31,567	52,318	1.69
2-axle, 6-tire	129,887	2,613	2,647	5,417	202,909	343,473	11.12
3-axle	27,309	225	297	1,083	17,461	46,375	1.50
All single-unit trucks	259,729	2,838	8,216	13,652	375,093	659,528	21.35
Vehicle combinations:							
Tractor-semitrailer	94,307	49,529	917	3,533	181,504	329,790	10.68
Truck-trailer	11,368	4,401	96	386	16,129	32,380	1.05
All combinations	105,675	53,930	1,013	3,919	197,633	362,170	11.73
All trucks and combinations	365,404	56,768	9,229	17,571	572,726	1,021,698	33.08
All vehicles	910,211	64,036	57,088	89,281	1,967,831	3,088,447	100.00

percent, of the total of $3,088 million of State motor-vehicle tax payments made during 1952. Registration-fee payments totaling $910 million brought in 29.5 percent; motor-carrier tax payments of $64 million provided 2.1 percent; operator- and chauffeur-license incomes provided $57 million, or 1.8 percent; and miscellaneous fees totaled $89 million, or 2.9 percent. The most natural comparison of total payments is that between passenger cars and other types of vehicles. Of the $3,088 million in State road-user taxes paid by all vehicles in 1952, $1,987 million was paid by passenger cars; $1,022 million was contributed by trucks and combinations; $59 million by buses. The remainder is accounted for by nearly $7 million assigned to motorcycles and $14 million assigned to camp, farm, and other light trailers.

Table 2 and figure 1 indicate that automobiles constituted 83.0 percent of motor-vehicle registrations in 1952 and accounted for 64.8 percent of the user taxes. Buses, relatively negligible in the gross totals, were approximately 0.3 percent of the numbers registered and paid 1.9 percent of the user-tax revenues. Trucks and combinations accounted for 16.8 percent of the vehicles and 33.3 percent of the revenues.

A somewhat different grouping of vehicles brings out the relation of numbers and payments more clearly. If the values for panels and pickups and other 4-tire trucks are added to those for automobiles, this constitutes what may be called the light-vehicle group. With this grouping, it is found that automobiles and light trucks formed 93.4 percent of the registered vehicles in 1952 and paid 73.6 percent of the road-user taxes. Medium and heavy trucks and combinations accounted for 6.3 percent of the vehicles and 24.5 percent of the user-tax payments. This finding is two-edged, in a sense. By the act of putting light trucks with passenger cars, the total of the truck contribution is diminished, but the weighting of payments in relation to numbers is increased from less than 2 to 1 to nearly 4 to 1.

Some of the figures for individual types in the visual classification are revealing. Two-axle, six-tire trucks amounted to 5.0 percent of the vehicles and their tax payments were 11.2 percent of the total. Three-axle trucks, constituting 0.3 percent of the vehicles, paid 1.5 percent of the revenues. Tractor-semitrailers, which added only 0.84 percent to the vehicle total, paid 10.8 percent of the total. Truck-trailer combinations constituted 0.08 percent of the vehicles and made 1.1 percent of the tax payments. Thus combinations as a group amounted to less than 1 percent of the vehicles but accounted for nearly 12 per cent of the revenues.

In average payments per vehicle during 1952, it is found that the value for automobiles was approximately $45.50, that for buses was $404, and that for trucks and combinations was slightly less than $116. Within the truck and combination group, there is found an average payment of $47 by panels and pickups and $59 by other 2-axle, 4-tire trucks; the general average for 2-axle, 4-tire trucks was $49. Two-axle, six-tire trucks paid, on the average, $130, and 3-axle trucks about $265. The average payment for combinations as a group was $745, $746 being the average for tractor-semitrailers, and $736 for truck-trailers. Too much should not be made of the comparison between the two types of combinations, because of the wide difference in both numbers and geographical distribution. In the regrouping of vehicles automobiles and light trucks are found to have made an average payment per vehicle of $46; the average for medium and heavy trucks and combinations was $227.

Comparisons on a vehicle-mile basis are also given in table 2 and are illustrated in figure 2. Here it is found that automobiles, which constituted 83.0 percent of the registrations in 1952, accounted for 80.9 percent of the traffic volume. This may be compared with their contribution of 64.8 percent to the total road user revenues. If again automobiles and light trucks are combined, it is found that this group contributed 89.1 percent of the vehicle miles and 73.6 percent of the revenues. Medium and heavy trucks and combinations accounted for 10.2 percent of the traffic volume and 24.5 percent of the revenues. Combinations taken alone provide an interesting comparison: They constituted 0.92 percent of the

Table 2.—Estimate of State highway-user taxes paid in 1952 by vehicles in different types and weight groups

Vehicle type	Motor vehicles registered [1]		Vehicle-miles traveled		Highway-user taxes paid [2]		Average rate of payment	
	Number	Distribution	Amount	Distribution	Amount	Distribution	Per vehicle	Per vehicle-mile
	Thousands	*Percent*	*Millions*	*Percent*	*1,000 dollars*	*Percent*		*Cents*
Passenger cars	43,654	82.96	409,271	80.89	1,987,279	64.78	$45.52	0.49
Buses	145	.28	3,564	.70	58,587	1.91	404.05	1.64
Single-unit trucks:								
Panel and pickup	4,629	8.80	33,971	6.72	217,362	7.08	46.96	.64
Other 2-axle, 4-tire	882	1.68	7,598	1.50	52,318	1.71	59.32	.69
2-axle, 6-tire	2,646	5.03	32,679	6.46	343,473	11.20	129.81	1.05
3-axle	175	.33	1,866	.37	46,375	1.51	265.00	2.49
All single-unit trucks	8,332	15.84	76,114	15.05	659,528	21.50	79.16	.87
Vehicle combinations:								
Tractor-semitrailer	442	.84	15,814	3.12	329,790	10.75	746.13	2.09
Truck-trailer	44	.08	1,197	.24	32,380	1.06	735.91	2.71
All combinations	486	.92	17,011	3.36	362,170	11.81	745.21	2.13
All trucks and combinations	8,818	16.76	93,125	18.41	1,021,698	33.31	115.87	1.10
All vehicles	52,617	100.00	505,960	100.00	3,067,564	100.00	58.30	.61
Regrouping of vehicle types:								
Passenger cars and light trucks [3]	49,165	93.44	450,840	89.11	2,256,959	73.57	45.91	.50
Medium and heavy trucks and combinations	3,307	6.28	51,556	10.19	752,018	24.52	227.40	1.46

[1] Publicly owned vehicles, motorcycles, and light trailers are excluded.
[2] Excludes $12,859,000 in fines and penalties, and tax payments of $14,117,000 assigned to light trailers and $6,766,000 assigned to motorcycles.
[3] Panels and pickups and other 2-axle, 4-tire trucks are grouped with passenger cars.

vehicles, traveled 3.4 percent of the vehicle-miles, and provided 11.8 percent of the revenues.

The final comparison shown in table 2 and figure 2 is that made on the basis of average road-user tax payments per mile of travel. The average payment by automobiles was 0.49 cent per vehicle-mile, or almost exactly one-half cent. Buses paid 1.64 cents per mile of travel and trucks and combinations, as a group, paid 1.10 cents. The average for all vehicles was 0.61 cent per mile of travel. When automobiles and light trucks are combined, the average payment per mile comes out exactly at one-half cent. Medium and heavy trucks and combinations, taken as a group, contributed 1.46 cents per vehicle-mile.

Among the general group of trucks and combinations, it is found that 2-axle, 4-tire trucks paid between 0.6 and 0.7 cent per mile of travel. Two-axle, six-tire trucks paid 1.05 cents per vehicle-mile, and 3-axle trucks 2.49 cents, the average for single-unit trucks being 0.87 cent. The rate per vehicle-mile for combinations as a group was 2.13 cents, tractor-semitrailers paying 2.09 cents and truck-trailer combinations 2.71.

In the interpretation of these figures it should be borne in mind that they are Nation-wide totals and averages derived by processing in various ways the data reported by 48 States and the District of Columbia, each of which has its own schedule of user taxes, with the rates of payment differing widely from State to State. The vehicles of each type and size group may contribute relatively more in one State and relatively less in another. The findings of this study summarize the situation as a whole, giving approximate values of the aggregate and average payments by each vehicle group, and thereby affording comparisons of the extent to which each group shares in the total burden of State road-user taxation.

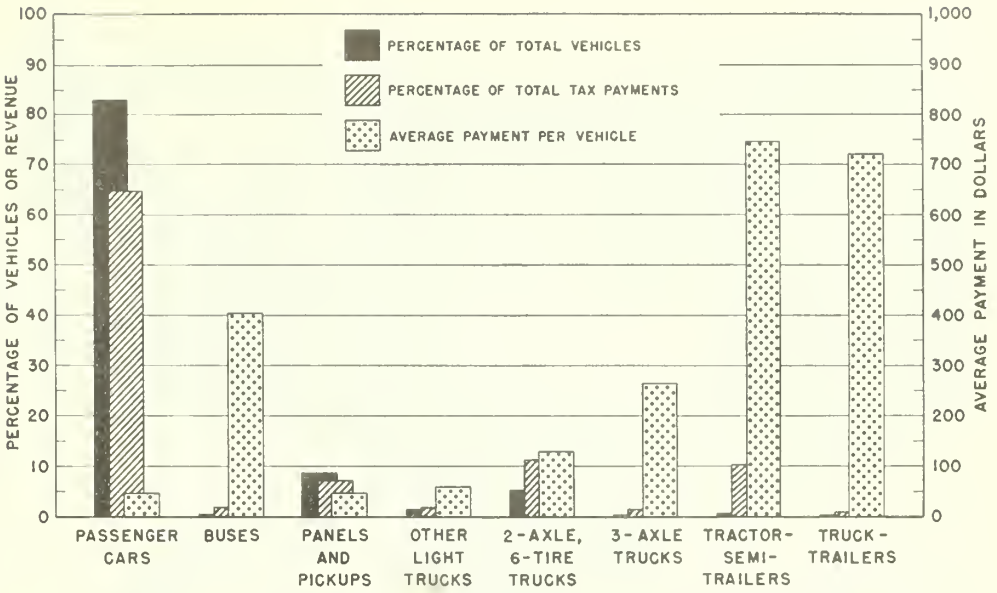

Figure 1.—Comparison of registrations and tax payments by vehicle types.

Vehicle Classifications

Gross-weight distribution

Although registrations and fee payments are segregated in State records by major types of vehicles, the further task of distributing numbers and fees among various groups of trucks is a complex matter. The differences among the various State bases of registration had to be reconciled, and, to do this, factors were developed for converting the available data supplied by the States to a gross-weight basis. Thirty-one States had furnished, for 1952, data on truck-weight or capacity groupings according to their own registration bases. In a few States this basis was the unrealistic "manufacturers' rated capacity," and in some it was on variations of net or empty weight, but for the majority, it was gross-vehicle weight. Some States use a combination of factors. Although more than half of the States now register trucks and combinations on the basis of gross weight, it can be seen in figure 3 that quite a few, including some of the larger ones, register on different bases. Conversion factors were estimated and, for each State for which data were available on some basis other than gross-vehicle weight, the conversion factors were applied to obtain an approximation of the State's registration according to the groups in which they would have fallen if all States required registration on a gross-vehicle-weight basis.

While there is no need at this point to outline those conversion factors in detail, here are some examples. Single-unit trucks of 4,500 pounds or less empty weight in States registering on an empty-weight basis were considered to be in the gross-vehicle-weight class of 1.8 times their empty weight; single-unit trucks in the 4,501–8,000-pound empty-weight group were considered to belong with vehicles of exactly twice their weight when registered on a gross-weight basis; and vehicles with an empty weight of more than 8,000 pounds were converted to gross-weight values of 2.5 times their empty weight. In States where tractor-trucks are registered on an empty-weight basis they were considered to represent combinations of 5 times the empty weight of the tractor alone; and tractors registered on a gross-weight basis were converted to gross combination weights of 1.8 times the gross registered weight of the tractor alone.

All in all, there were 18 States for which data were available on a gross-vehicle-weight basis, and it was possible to convert the data from an additional 12 States registering on other bases. However, in order to obtain balance, and because of questionable factors in the original material, data for 15 States were selected as representative. These 15 States registered more than 44.2 percent of all trucks in the United States in 1952. The percentages obtained from this 15-State sample were applied to national totals of trucks registered. This distribution is shown in table 3 (reading across) and in figure 4.

In 1952, the year on which this study is

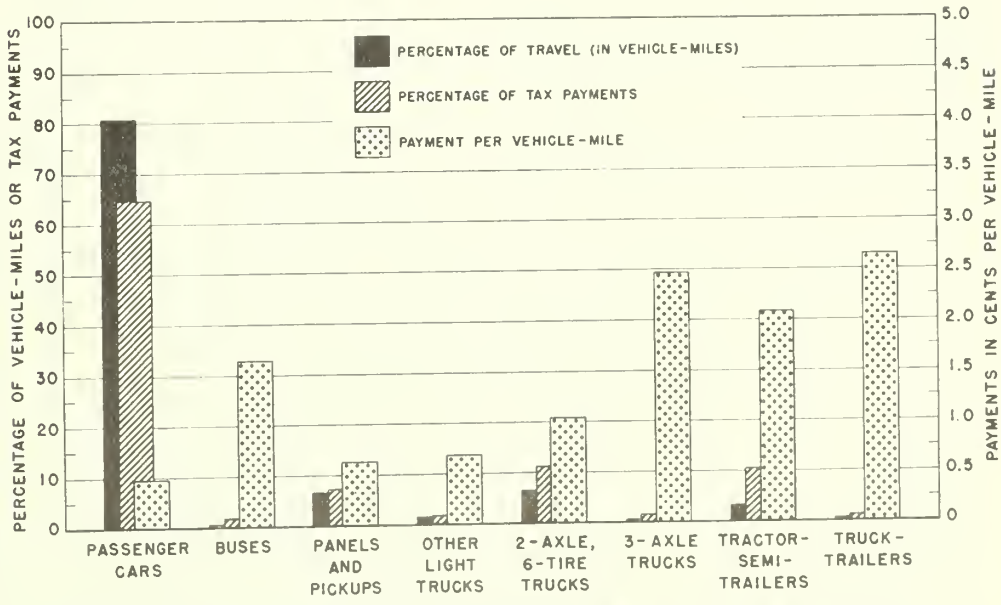

Figure 2.—Comparison of travel, tax payments, and payments per vehicle-mile.

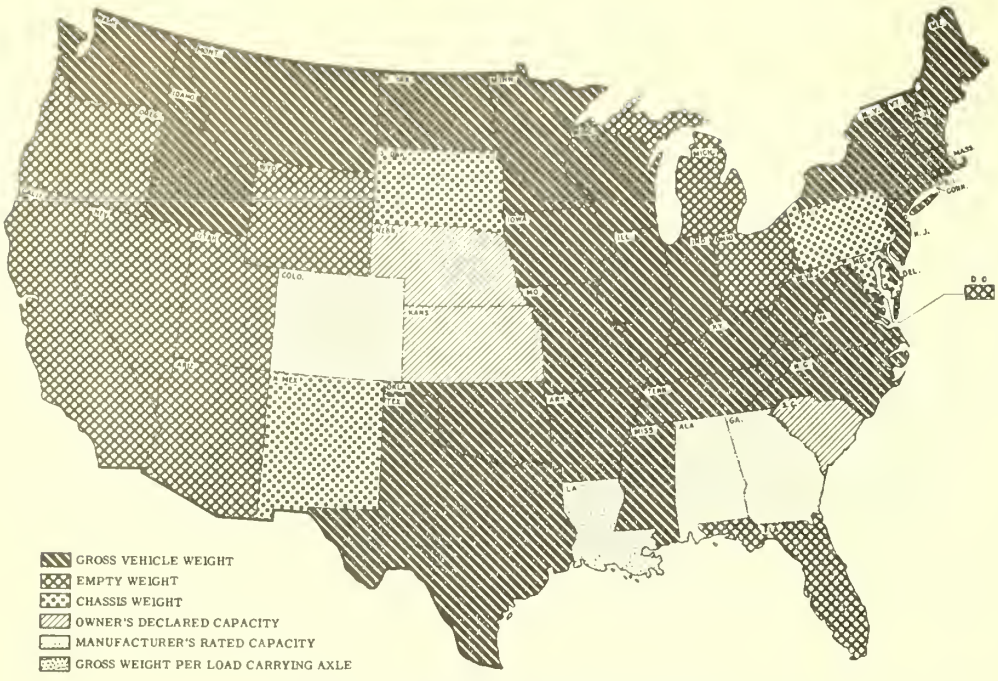

Figure 3.—Truck registration fee basis.

based, there were 8,818,000 trucks registered (excluding publicly owned vehicles). Of these, after converting to a gross-vehicle-weight basis, as described above, there were 5,679,000 in the 8,000 pounds and under group, or 64.4 percent. An additional 26.3 percent, or 2,318,000 were in the groups from 8,001 to 20,000 pounds. Only 370,000, or 4.2 percent, of the trucks were in the 20,001–30,000-pound range; and 212,000, or 2.4 percent, were between 30,001 and 40,000 pounds. The trucks and combinations of over 40,000 pounds accounted for 2.7 percent of the total, or 238,000 vehicles and combinations. Thus, only 9.3 percent of all trucks were more than 20,000 pounds gross weight.

Visual classification of vehicles

The previous discussion concerns the distribution of vehicles on registration bases, and some of the difficulties encountered in computing a uniform distribution on the basis of gross-vehicle or gross-combination weights. An entirely different problem arises in adapting the computed gross-vehicle-weight basis to the actual vehicles operating on the highway as they are observed from traffic counting or weighing stations. Determination of the taxes paid by various vehicles requires considerable knowledge of the mileages they travel; and this must be computed primarily from observation. Registration fees do not vary with the amount of travel. Motor-carrier taxes do vary to a considerable degree with the amount of travel, and fuel taxes paid vary in direct proportion as travel varies.

The "visual" classification of vehicles shown in table 3 (reading down) and in figure 5 is that ordinarily used in recording and publishing traffic-volume information. This was the principal reason dictating its adoption for this study, although another factor prompting its use was that this classification is more meaningful than is a classification based solely upon gross weight.

Although the visual classification is so commonly used in presenting traffic data, vehicles in use or registered cannot readily be classified on this basis. In spite of the fact that tractor-trucks or panels and pickups are

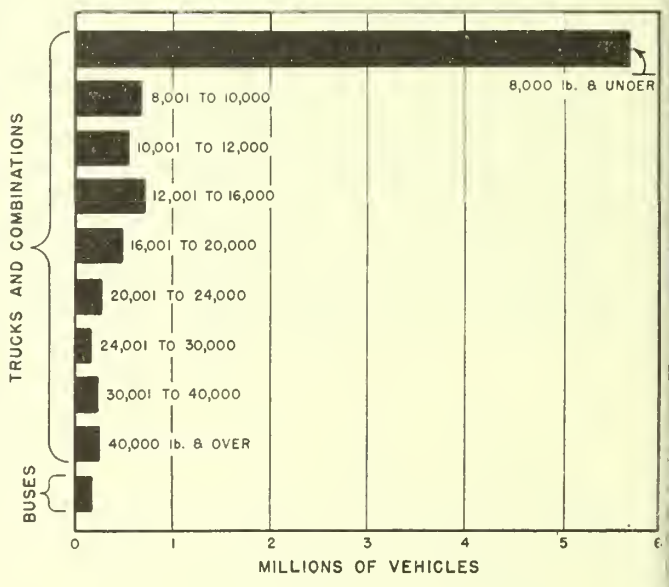

Figure 4.—Distribution of commercial vehicles by registered gross weight.

Table 3.—Estimated distribution of trucks and combinations in 1952 by visual classification and registered gross weight

Registered gross weight	Single-unit trucks								Vehicle combinations				Total	
	2-axle, 4-tire				2-axle, 6-tire		3-axle		Tractor-semitrailer		Truck-trailer			
	Panel and pickup		Other											
	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution
Pounds	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*
8,000 and under	4,497	51.000	706	8.000	476	5.400	-----	------	-----	------	-----	------	5,679	64.400
8,001–10,000	132	1.500	88	1.000	424	4.800	-----	------	-----	------	-----	------	644	7.300
10,001–12,000	-----	------	88	1.000	441	5.000	-----	------	-----	------	-----	------	529	6.000
12,001–16,000	-----	------	-----	------	656	7.440	17	0.200	29	0.330	2	0.030	704	8.000
16,001–20,000	-----	------	-----	------	385	4.360	26	.300	29	.330	1	.010	441	5.000
20,001–24,000	-----	------	-----	------	142	1.615	18	.200	73	.825	5	.060	238	2.700
24,001–30,000	-----	------	-----	------	60	.685	26	.300	37	.415	9	.100	132	1.500
30,001–40,000	-----	------	-----	------	62	.700	65	.735	82	.925	4	.040	213	2.400
Over 40,000	-----	------	-----	------	-----	------	23	.265	192	2.175	23	.260	238	2.700
Total	4,629	52.500	882	10.000	2,646	30.000	175	2.000	442	5.000	44	.500	8,818	100.000

registered separately in a few States, there is none in which the visual classification has been adopted in a general way as a basis for vehicle registration. Manufacturers' and trade-association statistics are no more helpful; manufacturers' gross-vehicle-weight rating has understandably become the basis upon which these groups publish most of their statistics on production and sales.

As a consequence, it became necessary in preparing the visual distribution of vehicles shown in table 3 and figure 5 to resort to other sources of information. One of these was the findings of the motor-vehicle-use studies conducted in five States, as presented in the project reports made on those studies. Another was the distribution of vehicles for seven urban areas reported in the home-interview samples taken in origin and destination studies made. A third was a report prepared on an analysis of the 1952 truck registrations in North Carolina.[2] Although none of these sources provided all of the information desired, it was possible by piecing this information together with that which was available from registration records in a few States to develop the distribution shown in table 3 and figure 5.

Some of these sources also provided gross-vehicle-weight distributions of individual visual classifications. With the help of these it was possible to calculate a cross-classification of vehicles by both visual and gross-weight classifications. This tabulation, table 3, provided a means of allocating registration and related fees and taxes according to both classifications.

Registration-Fee and Carrier-Tax Payments

Registration fees and related imposts

Total revenue from State registration fees and associated imposts amounted to $1,069,439,000, or $1,056,580,000 if the $12,859,000 of fines and penalties are excluded. Of this net amount, $910,211,000 were registration fees and the remainder of $146,369,000 was accounted for by title fees and taxes, transfer and reregistration fees, operators' and chauffeurs' licenses, and other miscellaneous allied revenue. Operators' and chauffeurs' licenses alone accounted for $57,088,000.

Registration fees

In order to allocate registration fees between the various principal groups of vehicles, average registration fees were computed from the basic data on which the previous study was based.[3] Although the present study deals in national totals, it is well to remember that there are great differences among the States in their taxation of motor vehicles. Property taxes on motor vehicles are not within the scope of this study, but there is considerable variation in their imposition and magnitude also.

The average registration fee for automobiles,

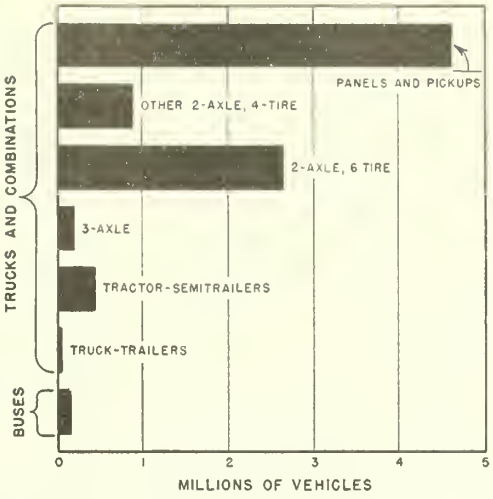

Figure 5.—Distribution of commercial vehicles by "visual" classification.

derived by simple division, is $11.81. The computed truck registration fees, derived by multiplying the numbers of vehicles in each group by the estimated average fees, yielded a total of $368,605,000, or not quite 0.9 percent more than the known total of $365,404,000. The average fees were therefore reduced 0.9 percent to arrive at the $365,404,000 total.

For 1952, truck and tractor registration fees totaled $320,251,000. To this amount was added the $59,270,000 of fees paid on various types of trailers and semitrailers, deducted from which was $14,116,000 estimated to have been paid on house trailers, light car trailers, etc. The resulting amount, $365,404,000, makes allowance for the fact that semitrailers and trailers are registered separately in many States, and that there are considerably greater numbers of semitrailers than tractors.

There were 5,679,000 trucks in the 8,000 pounds or less gross-vehicle-weight group. When converted to the visual classification, 4,497,000 fell into the panel and pickup group with 4 tires, 706,000 were other single-unit trucks with 4 tires, and 476,000 were 2-axle, 6-tire single-unit trucks. The total registration fees of these groups amounted to $103,417,000. It seems probable that the panels and pickups pay slightly smaller fees than the other vehicles in this group.

In this respect, it is interesting to note that a great many States impose lower registration fees on farm trucks than on vehicles not qualifying for that classification. The vast majority of farm trucks are in the pickup and other relatively light groups. To make allowance for this difference in fees, it was assumed that the average registration fee of the 706,000 4-tire single-unit trucks other than panels and pickups had a value of X and that the registration fee of panels and pickups had an average value of X minus 5 percent, and that the 2-axle, 6-tire vehicles in the group had a registration fee with the value of X plus 5 percent. The same technique was applied to the fees of the vehicles in the 8,001–10,000-pound group. For the 529,000 trucks in the 10,001–12,000-pound group, it was assumed that the 88,000 4-tire trucks had an average registration fee of 5 percent less than the 441,000 6-tire single-unit trucks in the group. A similar method was followed in distributing the registration fees of each of the gross-vehicle-weight classes to the visual classifications. In each instance, however, a heavier weighting factor was given to the registration fees for combinations when they fell in the same gross-weight group as single-unit trucks.

Miscellaneous fees

The allocation of operators' and chauffeurs' licenses had to be arbitrary. Some States do not require chauffeurs' licenses and others do not require ordinary operators' licenses of those who hold chauffeurs' licenses. The total chauffeurs' license fees attributed to truck operators was $9,229,000. It was assumed that one chauffeur's license at an average fee of $1.80 should be attributed to each vehicle in the gross registration weight classes of 20,000 to 40,000 pounds, and 1.5 chauffeurs' licenses should be attributed to each vehicle over 40,000 pounds. The remainder of the chauffeurs' licenses and the fees derived therefrom were attributed to trucks in the various groups under 20,000 pounds. Chauffeur license payments attributed to bus operators were computed as approximately 2 per vehicle or 290,000, and at $1.80 each these amounted to $522,000. Motorcycle operators' licenses were estimated at 25 cents per registered motorcycle, and amounted to $102,000. The remainder of operators' and chauffeurs' license payments, $47,235,000, was allocated to passenger-car operators.

After allocating operators' and chauffeurs' license revenues to various groups of vehicles, there remained $89,281,000 of miscellaneous fees to be assigned. This was done insofar as possible by examination of the individual State reports and allocating the fees to individual groups where possible. As a result of this examination of State reports, $17,571,000 was assigned to trucks. This amounted to $1.99 each. In this distribution, however, consideration was given to size and value of the vehicles since these factors affected the receipts. Title fees, transfer fees, and issuance fees were distributed to trucks on a numerical basis. Nonresident tag fees and a small amount of other miscellaneous fees were distributed between trucks on the basis of a sample distribution in five States drawn from the individual reports of the States in the Bureau of Public Roads files. The truck share of special titling taxes, amounting to $32,489,000, was distributed on the basis of gross vehicle weights, since these are ad valorem taxes and it seemed that there should be a high degree of correlation between value and weight. Undoubtedly this is susceptible of refinement, but it is probable that no great violence is done by this approach.

It was assumed that the miscellaneous revenues to be assigned to buses averaged the same as those assigned to trucks, i. e., $1.99 each, or a total of $289,000. Miscellaneous revenues of $1.00 each were attributed to the 407,000 registered motorcycles. The remaining miscellaneous fees,

[2] *Analysis of the 1952 registration of property-carrying vehicles in North Carolina.* Division of Statistics and Planning, North Carolina State Highway and Public Works Commission, Raleigh, 1953. Tables I and II, pp. 6 and 9.

[3] See footnote 1, p. 17.

$71,014,000, were attributed to automobiles, and amounted to $1.63 per automobile when the amount is divided by the number of registered vehicles.

Carrier taxes

The prior discussion has outlined the major phases of assigning registration and associated fees. The assignment of the $64,036,000 in motor-carrier tax revenues was made by study of the individual reports of the States. This indicated that $7,268,000 might be assigned to buses and the remaining $56,768,000 assigned to trucks. Undoubtedly there are some instances of certain carrier taxes or public-service permit fees and related revenues that may be attributed to taxicabs but insufficient evidence was found of such payments to make any allocation. In any case, it is improbable that a substantial amount would be involved.

For the purpose of this study it was also assumed that carrier taxes can be assigned entirely to buses and to trucks of more than 12,000 pounds gross-vehicle-weight rating. Since the individual State records did not distinguish between the classes of vehicles upon which carrier taxes were levied, an arbitrary procedure was adopted in assigning them to the various groups. By taking the average amount of motor-carrier tax that would be paid by a vehicle of over 40,000 pounds as the quantity X, it was assumed, in computing carrier taxes, that vehicles in the 30,001–40,000-pound-group could be assigned a value of $0.75X$; trucks and combinations in the 24,001–30,000-pound-group, a value of $0.5X$; vehicles in the 20,001–24,000-pound-group, a value of $0.25X$; vehicles in the 16,001–20,000-pound-group, a value of $0.1X$; and vehicles in the 12,001–16,000-pound-group, a value of $0.05X$. The value of X was found to be $94.32. It might be said that this is systematic guessing, and there would be more than a grain of truth to it. Yet, in the absence of detailed basic data, any assignment of motor-carrier taxes to various groups of vehicles must necessarily be on an arbitrary basis and, regardless of the complexity of any formula adopted, it would be reasonably certain to contain many of the properties of the estimate made here.

Travel and Fuel-Tax Payments

Although much is known about the character and extent of motor-vehicle use, there is a present lack of complete information about the distribution of highway travel in rural and urban areas, especially that pertaining to the subdivision of this travel among the classes of vehicles for which it was desired to make estimates in this study. Nevertheless, such an estimate of travel during 1952, classified according to these vehicle types, had to be made if the fuel use and fuel-tax payments of the individual types of vehicles were to be calculated.

Motor-vehicle travel

Estimates of passenger-car, bus, and truck travel in the United States were issued by the Bureau for each of the years from 1936 through 1948.[4] The principal factors controlling the calculations made for 1936 were the traffic volumes, characteristics, and relations as determined from rural traffic counts, and from the motor-vehicle allocation and road-use studies conducted between 1935 and 1939, covering both rural and urban travel.

These projects were included in the program of basic highway planning studies undertaken jointly by the State highway departments and the Bureau of Public Roads. Estimates for the succeeding years were based upon the calculations made for 1936, such modifications being made as were necessary to reflect known trends in motor-vehicle registrations, fuel consumption, and vehicle use. The principal factors controlling the calculations for the individual years were the annual estimates of rural traffic made by the Bureau from traffic counts obtained by the State highway departments, the annual reports of the highway use of motor fuel made by State authorities, and reports of motor-vehicle registrations, also made by State authorities. Publication of these estimates was discontinued after 1948 because it was felt that some of the basic relations existing in 1936, and upon which the entire structure of the estimates was predicated, might have changed considerably. Since that time only estimates of rural travel have been published.

Basis of Travel Estimates

The same basic procedures employed in preparing the estimates for 1936–48 were used in developing the estimate of the total passenger-car, bus, and truck travel for 1952, presented in table 4. For purposes of this study, however, it was necessary to subdivide the estimate of total truck travel into the various visual classifications shown in the table. In rural areas, classification counts have been made regularly by the State highway departments as a part of the highway planning survey operations, and the percentage distribution shown by these counts was used in subdividing the total rural vehicle-mileage of trucks. In urban areas, comprehensive classification count data are not available. Two other sources of information are available from the planning survey operations conducted by the States, however, and these were used in subdividing the total urban vehicle-mileage of trucks. Estimates of travel by the various visual classifications of trucks were developed for the large cities from information collected in origin and destination traffic studies of the home-interview type, and for the smaller cities from information obtained in motor-vehicle-use studies.

In the home-interview origin and destination studies it is standard practice to collect data concerning the type of truck, the licensed gross weight, and the daily mileage traveled in the urban area as well as the origin and destination of each trip. Information is also available in these studies concerning the number, type, origin, and destination of all trucks entering and leaving urban areas. Twelve cities[5] were selected from those in which home-interview studies have been made and special tabulations of the urban travel by type of truck were made for these cities. Some of these tabulations were made by the State highway departments and some by the Bureau of Public Roads. Percentages and factors developed from these data were used in estimating the urban vehicle-mileage of trucks by visual types in the larger cities for the country as a whole.

The motor-vehicle-use studies are also home-interview studies designed to obtain on a Statewide basis much the same types of information as are obtained for a single city or urban area in the home-interview origin-and-

[4] See previous annual articles on traffic in PUBLIC ROADS: vol. 25, Nos. 3, 7, 12, and vol. 24, No. 10.

[5] Camden, Dallas, Duluth, Houston, Madison, Minneapolis, Philadelphia, Racine, St. Paul, Seattle, Superior, and Washington, D. C.

Table 4.—Estimated travel during 1952, classified by place of travel [1] and by vehicle type

Vehicle type	Vehicle-miles of travel in—			Distribution of travel		
	Rural areas	Urban areas	Total	Rural areas	Urban areas	Total
	Millions	*Millions*	*Millions*	*Percent*	*Percent*	*Percent*
Passenger cars	213,464	197,404	410,868	77.01	83.98	80.21
Buses:						
Commercial	1,444	1,750	3,194	.52	.74	.62
Other	1,026	114	1,140	.37	.05	.22
All buses	2,470	1,864	4,334	.89	.79	.84
Single-unit trucks:						
Panel and pickup	22,075	13,324	35,399	7.97	5.67	6.91
Other 2-axle, 4-tire	2,083	5,834	7,917	.75	2.48	1.55
All 2-axle, 4-tire trucks	24,158	19,158	43,316	8.72	8.15	8.46
2-axle, 6-tire	20,453	13,600	34,053	7.38	5.79	6.65
3-axle	1,557	388	1,945	.56	.16	.38
All single-unit trucks	46,168	33,146	79,314	16.66	14.10	15.49
Vehicle combinations:						
Tractor-semitrailer	14,013	2,465	16,478	5.06	1.05	3.22
Truck-trailer	1,061	187	1,248	.38	.08	.24
All combinations	15,074	2,652	17,726	5.44	1.13	3.46
All trucks and combinations	61,242	35,798	97,040	22.10	15.23	18.95
All vehicles	277,176	235,066	512,242	100.00	100.00	100.00

[1] Urban areas include all incorporated places and delimited urban compacts.

Table 5.—Estimated travel during 1952, classified by ownership of vehicle and by vehicle type

Vehicle type	Vehicle-miles of travel by—			Distribution of travel by—		
	Total	Government-owned vehicles	Private and commercial vehicles	Total	Government-owned vehicles	Private and commercial vehicles
	Millions	*Millions*	*Millions*	*Percent*	*Percent*	*Percent*
Passenger cars	410,868	1,597	409,271	80.21	25.42	80.89
Buses:						
Commercial	3,194	------	3,194	.62	------	.63
Other	1,140	770	370	.22	12.26	.07
All buses	4,334	770	3,564	.84	12.26	.70
Single-unit trucks:						
Panel and pickup	35,399	1,428	33,971	6.91	22.73	6.71
Other 2-axle, 4-tire	7,917	319	7,598	1.55	5.08	1.50
All 2-axle, 4-tire trucks	43,316	1,747	41,569	8.46	27.81	8.21
2-axle, 6-tire	34,053	1,374	32,679	6.65	21.87	6.46
3-axle	1,945	79	1,866	.38	1.26	.37
All single-unit trucks	79,314	3,200	76,114	15.49	50.94	15.04
Vehicle combinations:						
Tractor-semitrailer	16,478	664	15,814	3.22	10.57	3.13
Truck-trailer	1,248	51	1,197	.24	.81	.24
All combinations	17,726	715	17,011	3.46	11.38	3.37
All trucks and combinations	97,040	3,915	93,125	18.95	62.32	18.41
All vehicles	512,242	6,282	505,960	100.00	100.00	100.00

estination studies. Because of their State-vide, rather than local emphasis, the sampling ates employed within cities in the motor-ehicle-use studies are much lower than those sed in the origin-and-destination studies; herefore, the stability and reliability of the notor-vehicle-use samples are lower when nly a single city or size-group of cities is onsidered. However, the data available from hese studies could be used to good advantage n estimating the travel of various classes of rucks and combinations in the smaller-sized ities and villages as a whole. Data obtained n seven States, the only ones in which motor-ehicle-use studies have been completed up o the present, were used in making these stimates. In addition to the travel data pplied, information obtained through these tudies relative to the distributions of dwelling nits, population, and motor vehicles was also sed in refining the calculations.

Other sources of information used included stimates of travel by commercial and other buses reported by the industry,[6] and estimates of automobile use reported by the Automobile Manufacturers Association.[7]

Total motor-vehicle travel on all roads and streets during 1952 was calculated to be 512 billion vehicle-miles, of which 411 billion, about 80 percent, was estimated to have been performed by passenger cars, 79 billion, nearly 16 percent, by single-unit trucks, 18 billion, somewhat more than 3 percent, by tractor-semitrailer and truck-trailer combinations, and 4 billion, nearly 1 percent, by buses.

This tabulation includes the travel of publicly owned non-military vehicles. It was desired to limit the calculation of fuel consumption and fuel-tax payments to the classifications of private and commercial vehicles shown in table 3 and figure 5. Consequently, the travel of publicly owned vehicles had to be eliminated from the estimated travel of all vehicles shown in table 4.

Estimates of the travel and fuel consumption of Federal civilian vehicles were determined from statistics compiled by the United States Bureau of the Budget, while estimates of the travel and fuel consumption of motor vehicles owned by State, county, and other local government agencies were developed from reports made (of the numbers of such vehicles) by most of the State highway departments to the Bureau of Public Roads.

The travel of publicly owned vehicles was determined to be 6 billion vehicle-miles, of which the amounts contributed by the individual vehicle types were as shown in the second column of table 5. The total travel of private and commercial motor vehicles, after deduction of public-vehicle travel, was 506 billion vehicle-miles, of which 409 billion was performed by passenger cars, 76 billion by single-unit trucks, 17 billion by combinations of freight-carrying vehicles, and nearly 4 billion by buses. The percentage distribution of this travel by vehicle groups was practically the same as for the total travel of all public, private, and commercial vehicles. This distribution is shown in figure 6.

Vehicle Operating Characteristics

In order to estimate the fuel consumption and fuel-tax payments for the individual classes of vehicles used in this study, it was necessary to determine certain of their operating characteristics such as average gross weights, percentages of vehicles using fuel other than gasoline, and rates of fuel consumption.

Operating gross weights

The calculated average operating gross weights used in this study for each type of vehicle are shown in table 6 and figure 7. Different methods were employed in arriving at the weights adopted for the various classes of vehicles.

The average operating gross weight of passenger cars was determined by a complex method of calculation in which these vehicles were divided by makes roughly into four groups, according to the weight of the most popular four-door sedan of each make. An average operating road weight was calculated for each make by adding to the shipping weight of the four-door sedan an allowance to cover nonstandard equipment (such as radios and heaters), fuel, water, two passengers, and baggage. The allowances varied from 600 pounds in the case of the vehicles in the lightest group to 900 pounds in the case of the heaviest vehicles. It was assumed that vehicles of all weight groups would have the same average travel. The average operating

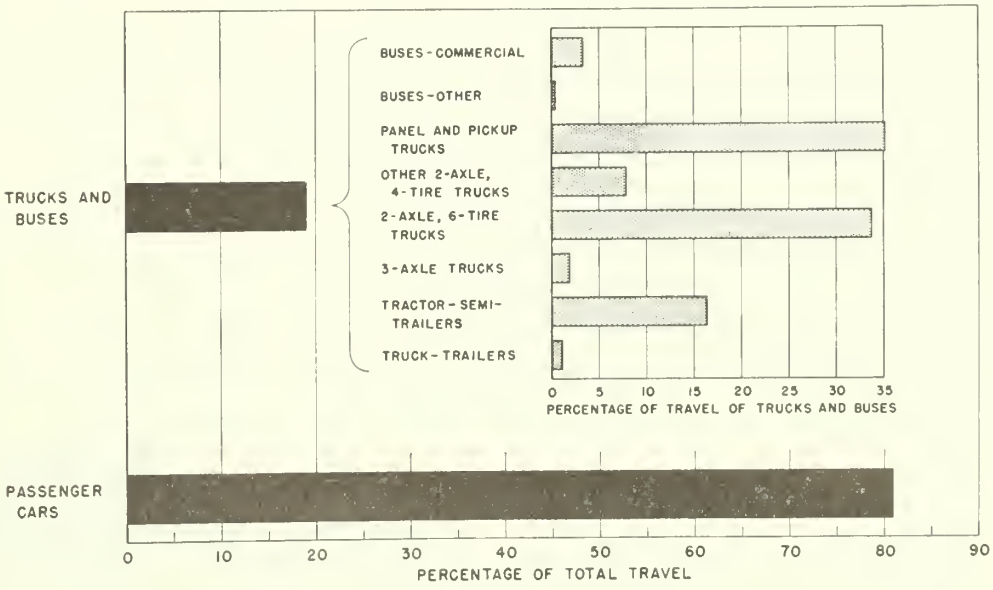
BUSES - COMMERCIAL
BUSES - OTHER
PANEL AND PICKUP TRUCKS
OTHER 2-AXLE, 4-TIRE TRUCKS
2-AXLE, 6-TIRE TRUCKS
3-AXLE TRUCKS
TRACTOR - SEMI-TRAILERS
TRUCK - TRAILERS
TRUCKS AND BUSES
0 5 10 15 20 25 30 35
PERCENTAGE OF TRAVEL OF TRUCKS AND BUSES
PASSENGER CARS
0 10 20 30 40 50 60 70 80 90
PERCENTAGE OF TOTAL TRAVEL

Figure 6.—Percentage distribution of travel by private and commercial vehicles in 1952.

[6] *Pustransportation*, vol. 32, No. 2, Feb. 1953, McGraw-Hill Publishing Co., New York, N. Y.

[7] *Automobile facts and figures*, 31st ed., 1951, Automobile Manufacturers Association, Detroit, Mich. See especially p. 48.

Table 6.—Operating characteristics of various types of motor vehicles

Vehicle type	Average operating gross weight	Distribution of travel according to type of fuel used			Rates of fuel consumption by type of fuel used		
		Gasoline	Diesel	Other	Gasoline	Diesel	Other
	Pounds	*Percent*	*Percent*	*Percent*	*Gallons per mile*	*Gallons per mile*	*Gallons per mile*
Passenger cars	3,965	100.0	(1)	(1)	0.06704	-------	-------
Buses:							
Commercial	23,000	39.1	55.9	5.0	.26870	0.18590	0.26690
Other	11,600	100.0	(1)	(1)	.12540	-------	-------
Single-unit trucks:							
Panel and pickup	4,639	100.0	(1)	(1)	.07350	-------	-------
Other 2-axle, 4-tire	5,834	100.0	(1)	(1)	.08420	-------	-------
2-axle, 6-tire	11,684	100.0	(1)	(1)	.12590	-------	-------
3-axle	23,611	100.0	(1)	(1)	.18980	-------	-------
Vehicle combinations:							
Tractor-semitrailer	35,602	86.5	12.6	.9	.24120	.17230	.26800
Truck-trailer	46,885	86.5	12.6	.9	.28320	.20230	.31470

[1] Percentage negligible.

gross weight for all passenger cars was calculated to be 3,965 pounds.

The operating characteristics of commercial buses differed so greatly from those of other types of buses that they were treated separately from the other types, such as privately owned buses operated by schools or institutions. The operating gross weight of 23,000 pounds assigned to commercial buses was determined by adding to the curb weight of a typical 42-passenger bus, such as is used in either city or suburban service, the weight of a load of 21 passengers. The operating gross weight of 11,600 pounds assigned to other buses represents the combination of the curb weight of a typical medium-sized school bus and the weight of an average load of 20 children.

The weights shown for the various classes of trucks and combinations are averages obtained from loadometer studies conducted in 1952 by the State highway departments. A total of 134,564 vehicles as found in the traffic stream on main rural roads were weighed. Some were empty, some overloaded, and some only partially loaded. The weights reported reflect these conditions. Since no data were available on weights of vehicles operating in cities, the rural road weights had to be applied to all traffic.

Use of special fuels

Although the use of fuels other than gasoline in the propulsion of motor vehicles is increasing rapidly, the amount of such so-called "special" fuels used is still a relatively small percentage of the total fuel consumed on the highways. In 1952 the total of all motor fuel used in the United States was 40,584 million gallons, while the total amount of special fuels used for highway purposes was only 805 million gallons. This relatively small segment of motor-fuel consumption assumes greater importance, however, when it is considered that nearly all of this fuel is consumed by the larger commercial vehicles—buses and combinations of freight-carrying vehicles.

Information reported by the commercial bus industry indicates that very large portions of its operations are now carried on with buses propelled by diesel fuel, liquefied petroleum gas, and other nongasoline fuels. The specific percentage relations used in this analysis are based upon reports from 24 intercity, intracity, and suburban operators.[8]

These data, which appear to be supported by other reliable information, indicate that more than 50 percent of the fuel now used in common-carrier buses is diesel fuel, while the use of liquefied petroleum gas has become an important factor in some instances. Although there is undoubtedly some use of these fuels in buses engaged in other types of operations, available information seems to indicate that such use is insignificant.

Nongasoline fuels are also used to some extent in single-unit trucks, but, inasmuch as the achievement of significant savings from the use of these fuels requires large-scale operations, such use is thought to be negligible and all of the consumption of these fuels in freight-carrying vehicles was assigned to combinations rather than single vehicles.

Fuel consumption rates

The rate at which a certain motor vehicle or combination of vehicles will consume fuel in its operations over the highways is affected at any given time by a number of factors, among which the following are of major importance: Type and grade of fuel used, characteristics of the engine, gear ratios, frequency of stops, condition of the vehicle, gradients encountered, types and conditions of roads traveled, weather, operating gross weight of vehicle (or combination) and contents, and driving techniques employed.

When the total of all motor vehicles in service, operating throughout the year under widely varying conditions, is being considered, and if only a broad and general analysis is undertaken as was the case in this instance, the effects of such factors as frequency of stops, topography, weather, condition of the vehicle, and driving techniques employed tend to become compensating and have little effect upon the determination of average rates of fuel consumption. Consequently, in the analysis undertaken for this study, no attempt was made to take any factor other than gross vehicle weight into account except in a very limited way, as is noted subsequently.

Figure 8 shows the compromise curve indicating the relation between gross weight and gasoline consumption that was plotted from the equation developed for this article, and the other fuel-consumption data that were considered in developing it. This equation is intended to indicate approximate gasoline-consumption rates for vehicles with gross weights up to at least 72,000 pounds, operating under average conditions.

This gasoline-consumption equation was not statistically developed in the ordinary sense. Rather, it is a composite of values for numerous gross-weight groups obtained from each of several previous determinations by other investigators. Since it was beyond the scope of this study to assemble original data on the fuel-consumption rates of motor vehicles, it was necessary to draw on the work of others. Although many sources of data were investigated, none was found which appeared to meet present needs in all respects.

Some, like the determinations of the

[8] *Bus transportation*, vol. 32, No. 11, Nov. 1953, McGraw-Hill Publishing Co., New York, N. Y.

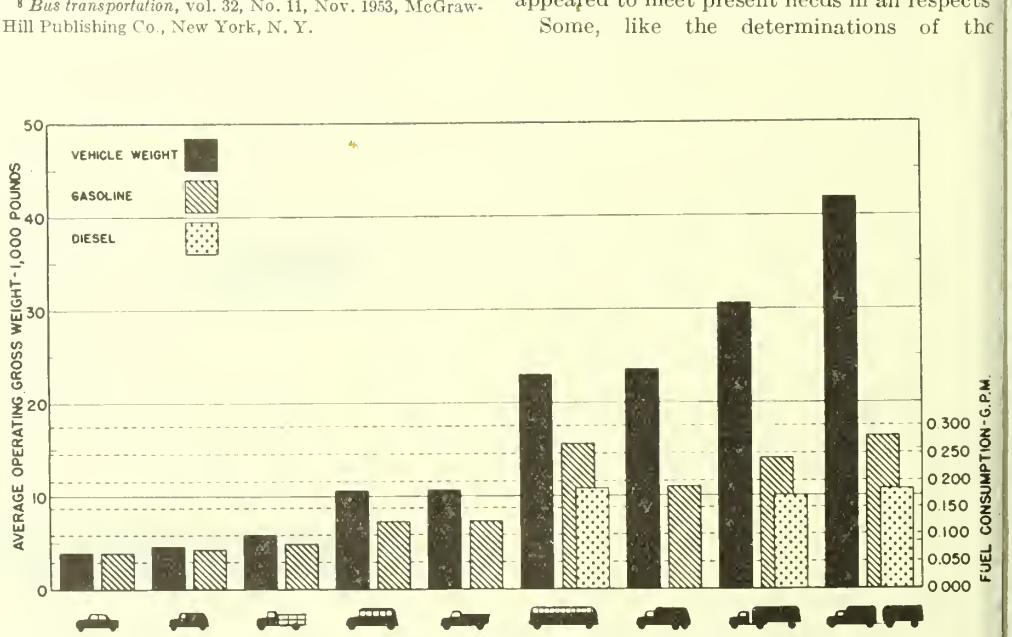

Figure 7.—Operating characteristics of various types of vehicles.

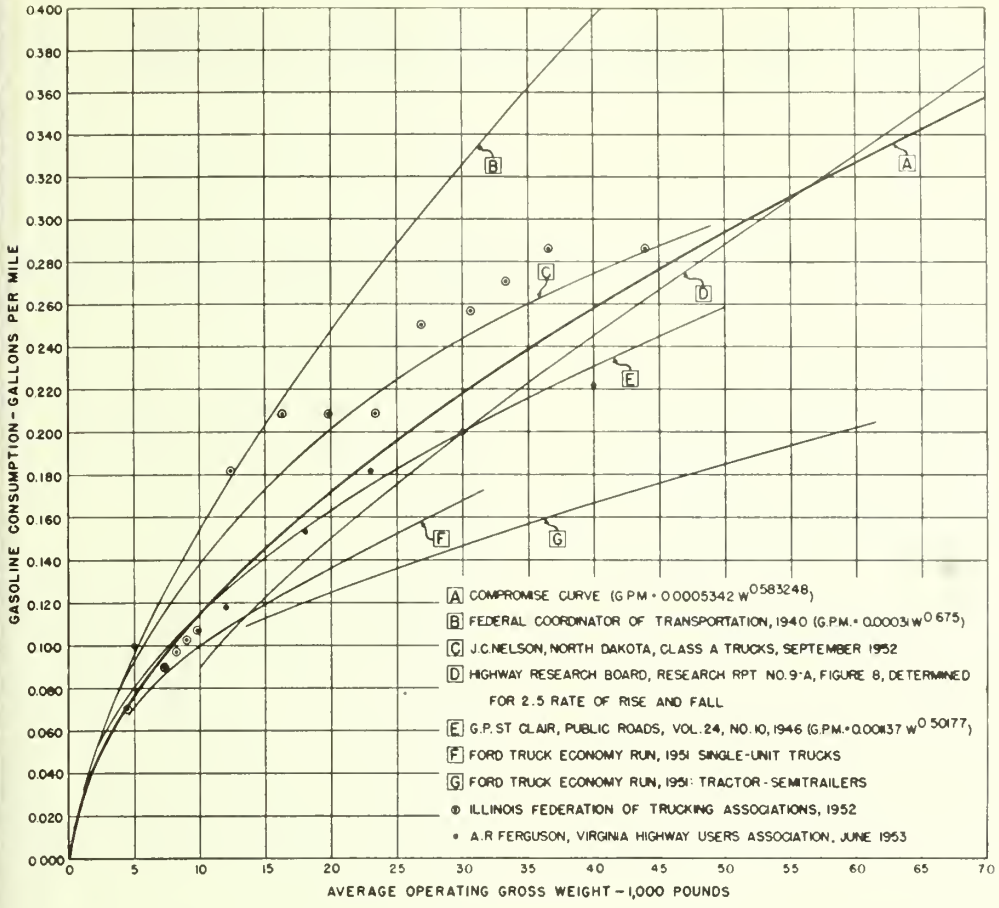
GASOLINE CONSUMPTION - GALLONS PER MILE
AVERAGE OPERATING GROSS WEIGHT - 1,000 POUNDS
A COMPROMISE CURVE (G.P.M. = 0.0005342 W^0.583248)
B FEDERAL COORDINATOR OF TRANSPORTATION, 1940 (G.P.M. = 0.0003 W^0.675)
C J.C. NELSON, NORTH DAKOTA, CLASS A TRUCKS, SEPTEMBER 1952
D HIGHWAY RESEARCH BOARD, RESEARCH RPT NO. 9-A, FIGURE 8, DETERMINED FOR 2.5 RATE OF RISE AND FALL
E G.P. ST CLAIR, PUBLIC ROADS, VOL. 24, NO. 10, 1946 (G.P.M. = 0.00137 W^0.50177)
F FORD TRUCK ECONOMY RUN, 1951 SINGLE-UNIT TRUCKS
G FORD TRUCK ECONOMY RUN, 1951 TRACTOR-SEMITRAILERS
ILLINOIS FEDERATION OF TRUCKING ASSOCIATIONS, 1952
A.R. FERGUSON, VIRGINIA HIGHWAY USERS ASSOCIATION, JUNE 1953

Figure 8.—Estimated fuel consumption of gasoline-powered vehicles compared with average operating gross weight.

calculated and plotted to logarithmic scales it was found that they fitted closely a straight line having the following equation:

$$G.P.M. = 0.0005342\,W^{0.583248}$$

where $G.P.M.$ = gallons per mile, and W = average operating gross weight of vehicle.

Consequently, it was decided that for purposes of the present analysis this fuel consumption equation could be applied throughout the entire range of gross weights for which gasoline consumption would need to be calculated.

As stated previously, this equation applies only to gasoline-powered vehicles. It is known that different rates of fuel consumption will apply to diesel-powered vehicles, but there are not sufficient data at hand to permit the calculation of an equation for them. After consultation with representatives of the trucking industry it was decided to assume that, for operating gross weights above 20,000 pounds, diesel vehicles will consume, on the average, about 30 percent less fuel than will gasoline-powered vehicles of equal weight. No special allowance was made for vehicles using other fuels, such as liquefied petroleum gas, partly because of their negligible importance in the nationwide picture, and partly because available data seemed to indicate that such vehicles generally have fuel-consumption rates closely approximating those of similar gasoline-powered vehicles.

All of the gasoline-consumption rates shown in table 6 and figure 7 were developed by applying the derived equation to the average operating gross weights shown except in the Federal Coordinator of Transportation,[9] were developed from information that is now so old that is does not reflect conditions now known to prevail, especially in the higher gross-weight brackets. Others, like the fuel-consumption rates developed from the Ford data,[10] are based upon limited coverage of engines, vehicle types, or loadings, and so tend to give values, for certain weight ranges, that deviate rather widely from the consensus of findings.

After plotting all of this information as shown in figure 8, it became evident that a new curve, or set of curves, should be developed. Some students of the problem contend that a single fuel-consumption curve cannot be developed to fit all types of vehicles from passenger cars through the heaviest combinations. When the gasoline-consumption equation adopted for use in this study was developed, it had not been predetermined that a single curve could be applied to all gross weights. However, when average fuel-consumption rates for each of numerous values of operating gross weights, ranging from 3,000 to 50,000 pounds, had been

[9] *Public aids to transportation,* Federal Coordinator of Transportation, vol. IV, 1940. See especially pp. 141–43.

[10] *Mileage, gross vehicle weight, and fuel consumption of commercial vehicles,* presented by Robley Winfrey at 33d Annual Meeting of the Highway Research Board, Washington, D. C., Jan. 12, 1954.

Table 7.—Fuel consumption and tax payments in 1952, classified by various types of private and commercial motor vehicles

Vehicle type	Total miles traveled	Gasoline-powered vehicles		Diesel-powered vehicles		Vehicles powered by other fuels		Total fuel consumed		Total tax payments
		Mileage	Fuel consumed	Mileage	Fuel consumed	Mileage	Fuel consumed	Gallons	Distribution	
	Millions	*Millions*	*Million gallons*	*Millions*	*Million gallons*	*Millions*	*Million gallons*	*Millions*	*Percent*	*Million dollars*
Passenger cars	409,271	409,271	27,438	----	----	----	----	27,438	68.771	1,353.3
Buses:										
Commercial	3,194	1,249	336	1,785	332	160	43	711	1.782	35.0
Other	370	370	46	---	---	---	---	46	.115	2.3
All buses	3,564	1,619	382	1,785	332	160	43	757	1.897	37.3
Single-unit trucks:										
Panel and pickup	33,971	33,971	2,497	---	---	---	---	2,497	6.259	123.2
Other 2-axle, 4-tire	7,598	7,598	640	---	---	---	---	640	1.604	31.5
All 2-axle, 4-tire trucks	41,569	41,569	3,137	---	---	---	---	3,137	7.863	154.7
2-axle, 6-tire	32,679	32,679	4,114	---	---	---	---	4,114	10.311	202.9
3-axle	1,866	1,866	354	---	---	---	---	354	.887	17.5
All single-unit trucks	76,114	76,114	7,605	---	---	---	---	7,605	19.061	375.1
Vehicle combinations:										
Tractor-semitrailer	15,814	13,679	3,299	1,993	343	142	38	3,680	9.223	181.5
Truck-trailer	1,197	1,035	293	151	31	11	3	327	.820	16.1
All combinations	17,011	14,714	3,592	2,144	374	153	41	4,007	10.043	197.6
All trucks and combinations	93,125	90,828	11,197	2,144	374	153	41	11,612	29.104	572.7
All vehicles	505,960	501,718	39,017	3,929	706	313	84	39,807	99.772	1,963.3
Fuel consumed by motorcycles, etc.	----	----	---	---		---	---	91	.228	4.5
Total fuel consumed and tax payments	----	----	---	---	---	---	---	39,898	100.000	1,967.8

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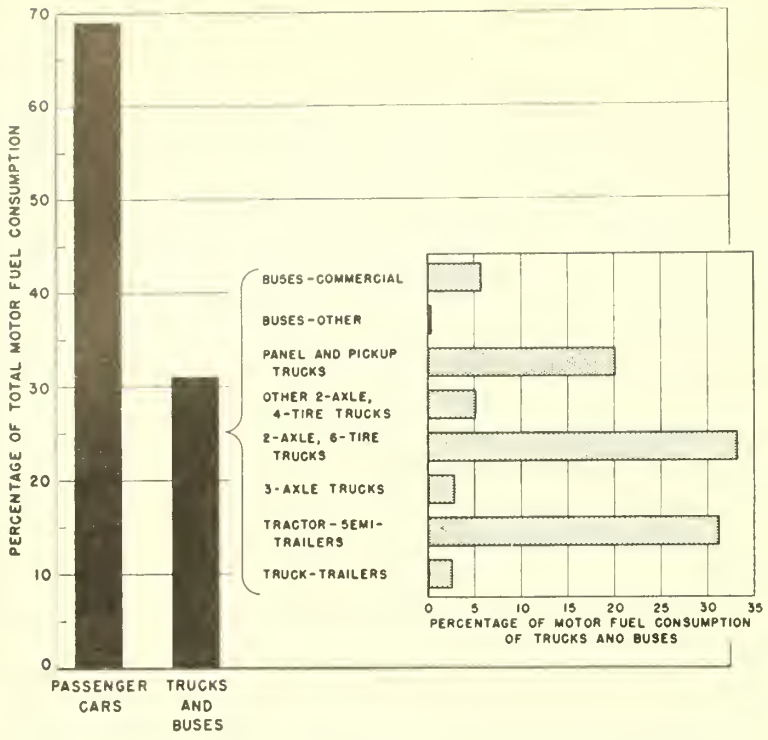

Figure 9.—Percentage distribution of motor-fuel consumption by private and commercial vehicles in 1952.

case of commercial buses. Available operating data indicate that relations between the gasoline-consumption rates and average operating weights of intercity buses are almost in line with the corresponding relations calculated by use of the equation, but that in the case of intracity and suburban buses the rates are much higher, probably because of the combined effects of frequent stops, urban congestion, and other factors peculiar to such operations. The composite gasoline-consumption rate shown was developed from operating statistics of the 24 companies previously cited.[11]

Fuel Consumption and Fuel-Tax Payments

Table 7 presents the calculated fuel consumption and fuel-tax payments of each of the various classes of vehicles indicated in the visual classification adopted for this study. Figure 9 shows the percentage distribution of indicated total fuel consumption.

Fuel consumption

The fuel-consumption data shown in table 7 were calculated by multiplying the total mileages traveled by the corresponding rates shown in table 6. Separate calculations of gasoline, diesel, and other fuel used were made on the basis of the percentages of total use there indicated.

The total calculated consumption of 39,807 millions of gallons of fuel of all kinds is 91 million gallons, or 0.225 percent, below the 39,898 million gallons of fuel used by private and commercial vehicles for highway purposes in 1952. However, the analysis made for this article did not take into account fuel consumed by motorcycles, motor scooters, and other similar vehicles, nor did it give consideration to the use of fuel on which highway-user taxes were paid and no refunds claimed for such nonhighway purposes as the operation of gasoline-powered lawnmowers, garden tractors, or small boats.

There were about 408,000 private and commercial motorcycles, motor scooters, and similar vehicles registered in 1952. If it can be assumed that these vehicles consumed an average of 200 gallons of fuel each during the year, their total consumption would have been nearly 82 million gallons, a not unlikely figure. Other investigators have averaged the annual consumption of motorcycles at 250 gallons, or even more. The Federal Coordinator of Transportation used a fuel-consumption rate of 0.027041 gallon per mile and an average annual mileage of 15,000 in estimating motorcycle fuel consumption in 1932.[12]

Fuel tax payments

During 1952, $1,968 million was collected from State taxes on motor fuel used for highway purposes. This total excludes taxes refunded upon nonhighway use of motor fuel and allowance made in a few States to taxpayers for cost of tax collection. It includes the incomes from certain miscellaneous receipts, such as distributors' and retailers' license fees, inspection fees, etc.

The total motor-fuel consumption covered by these tax payments is not exactly the same as the total of highway motor-fuel consumption by private and commercial vehicles of 39,898 million gallons. Tax collections do not indicate actual fuel consumption during a given year because of the time lag between payment of the fuel tax and the actual use of motor fuel and the handling of tax refunds for nonhighway use. Claims for nonhighway use in the fall of one year may not be paid until the following year.

For this reason it was decided not to attempt to calculate tax payments directly from the gallonage distribution shown in table 7. Instead, a percentage distribution was calculated from these data and applied to the total collections of $1,968 million, on the assumption that the percentages of use reflected by the collections would be essentially the same as those reflecting actual use during 1952. The results of this calculation are shown in the last column of table 7.

[11] See footnote 8, p. 24.

[12] See footnote 9, p. 25.

Driver Performance on Horizontal Curves

Reported by ASRIEL TARAGIN
Highway Engineer

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Some important observations were derived from this study of free-moving passenger-car speeds on horizontal curves of two-lane rural highways. Drivers do not slacken speed after entering a curve, presumably making their adjustments on the approaches. Speeds in the shorter-radius inside lanes are no less than in the outside lanes, despite the fact that sight distances average 20 percent shorter. Operating speed and the degree of curvature are linearly related: while drivers do not move at the high speeds permissible by the design on easy curves, they often exceed the design speed on sharp curves.

Superelevation apparently has no effect on vehicle speeds, and the utilized coefficient of side friction is relatively small when the superelevation is high. However, there is a close relation between "unit superelevation"—the rise per foot of width per degree of curvature—and safe driving speed. Few vehicles exceed a safe speed on horizontal curves designed with a unit superelevation of more than 0.005 foot per foot of pavement per degree of curvature.

Where the minimum sight distance is 400 feet or more, drivers travel at speeds from which a stop can be made within the available sight distance, but where the sight distance is below 400 feet, drivers travel at speeds from which a stop can be made within the available sight distance only when the perception and reaction times are ignored. Speeds are considerably lower on horizontal curves than on vertical curves with the same minimum sight distances.

It is evident that if roads are to be built safely for the accommodation of actual driver behavior—and this seems logical—the design of horizontal curves on two-lane rural highways should take into account unit superelevation, and should provide a minimum sight distance of at least 400 feet, allowing for reasonable perception, reaction, and braking time. That an adequate margin of safety is needed is evidenced from the implication in the study results that driver speeds are more controlled by the obvious danger of running off the road, when negotiating a horizontal curve, than by the possible hazard of an unseen object where sight distance is limited on either horizontal or vertical curves. The physical stress of centrifugal force can actually be felt, while the sight-distance hazard can only be imagined.

EXISTING highways are conglomerations of varied geometric designs. Some sections are designed in accordance with the most modern standards, to accommodate large volumes of traffic at relatively high speeds, but these are by far in the minority. The greater part of our highway system is of two-lane design, often inadequate for the volume of traffic carried. In many locations the most common deficiency is insufficient sight distance for safe operation at desired speeds on vertical and horizontal curves.

The effect of vertical curves on driver behavior and vehicle speeds was discussed in a paper presented by B. A. Lefeve at the annual meeting of the Highway Research Board in January 1953.[1] The problem of horizontal curvatures and their effects on driver performance is presented in this report.

Study Procedure

Driver performance and passenger-car speeds were recorded on a number of horizontal curves with minimum sight distances ranging from 200 to 655 feet and with curvatures from 3 to 29 degrees. Study locations were confined to sections of two-lane highway on which it might be expected that driver performance would be affected by horizontal curvature, superelevation, or limitation of sight distance. In no case did the approach grade exceed 3 percent, and no section had a vertical curvature in combination with the horizontal curvature.

All locations were on rural highways removed from the influence of intersections and with a minimum of interference from roadside development. The study included only passenger cars and, to insure that they were not influenced by other vehicles traveling in the same direction, those following another vehicle within a time spacing of 6 seconds were excluded.

The study was conducted in two phases. The first phase was initiated in 1951 in cooperation with the New York State Department of Public Works and included studies of driver behavior on 15 horizontal curves. At these curves the speed of each vehicle observed was recorded at 100-foot intervals over a distance of 1,000 feet, starting 500 feet ahead of the centers of the curves and ending 500 feet beyond the centers. This included the entire lengths of the curves, since they were from 400 to 900 feet long. (Observations of vehicle speeds on the approaches, however, were not included in this study.) The data were obtained in such a manner that the variation in speed on each curve could be related to the variation in the sight distance on the curve. The results obtained by this phase of the study are discussed in the first part of this report.

To supplement the data obtained in New York, the second phase of the study made use of data from studies conducted at 20 locations in Maryland, Illinois, Minnesota, and South Carolina. This phase consisted of determining passenger-car speeds at one point on each curve—at the point of minimum sight distance. The data collected at these locations were combined with the New York data for the greater portion of this report. The data from each study site were analyzed separately for vehicles traveling in the inside lane of the curve and for vehicles traveling in the outside lane of the curve.

It should be understood, of course, that the curves studied are on two-lane highways, and the "inside" lane is that side of the road nearest the center of the circle of which the curve is an arc; the "outside" lane, carrying traffic in the opposite direction, is the other half of the road.

The inside lane of a curve has a slightly sharper curvature and shorter radius than the outside lane. This difference should be kept in mind in the consideration of the results because the curvatures as reported are those as measured to the centerline of the pavement. Sight-distance measurements were made separately for each direction of travel, at the center of the lane, from a height of 4½ feet to an object 4 inches high in the same lane.

The speeds of approximately 125 free-moving passenger cars (not meeting another vehicle and more than 6 seconds behind the preceding vehicle) were observed at each study site. Satisfactory data were obtained for the inside lanes of 35 different curves and for the outside lanes of 33 of these curves, involving

[1] *Speed characteristics on vertical curves*, by B. A. Lefeve. Highway Research Board Proceedings, 1953, vol. 32, p. 395.

Table 1.—Locations studied and observed speed data on horizontal curves on two-lane highways

Location		Physical features			Inside lane of curve				Outside lane of curve			
						Speed at minimum sight distance				Speed at minimum sight distance		
State	Site No.	Pavement width[1]	Curvature	Superelevation	Minimum sight distance	Average	90-percentile	95-percentile	Minimum sight distance	Average	90-percentile	95-percentile
		Feet		*Ft./ft.*	*Feet*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*Feet*	*M.p.h.*	*M.p.h.*	*M.p.h.*
Ill.	1	18	25° 00′	0.083	200	23.7	28.9	29.4	300	22.6	28.5	29.4
N. Y.[2]	2–*I*	22	17° 30′	.062	215	36.5	44.0	45.0	---	---	---	---
N. Y.[2]	2–*O*	22	4° 45′	.062	---	---	---	---	220	38.0	43.0	46.5
Md.	3	[3] 26	23° 50′	.077	215	31.5	36.2	38.1	455	32.2	36.2	37.8
N. Y.	4	17	20° 30′	.083	220	28.0	34.5	36.0	220	30.0	35.0	37.2
Md.	5	[3] 28	6° 52′	.036	236	38.6	46.3	47.6	452	36.9	42.1	43.9
Md.	6	24	9° 10′	.042	256	41.4	47.5	49.1	453	40.7	47.1	49.5
Md.	7	[4] 22	12° 15′	.045	285	35.7	41.5	42.9	391	35.2	40.4	42.9
Md.	8	22	13° 44′	.068	297	40.4	49.7	52.9	323	41.2	46.9	48.6
Minn.[5]	9	26	29° 00′	.062	300	25.1	29.7	31.5	---	---	---	---
N. Y.	10	24	10° 00′	.073	300	35.5	44.0	46.4	325	37.5	44.5	45.5
Md.	11	24	3° 50′	.014	303	46.6	55.1	58.5	526	46.3	53.0	55.5
Md.	12	24	4° 35′	.028	303	40.8	46.5	49.0	489	42.4	47.2	50.4
Md.	13	[3] 26	10° 18′	.038	308	41.9	46.3	47.1	377	37.0	44.2	46.5
N. Y.	14	23	9° 30′	.073	320	37.8	45.0	47.6	320	37.0	44.2	46.3
N. Y.	15	22	11° 00′	.073	320	36.0	45.0	46.8	340	35.0	42.0	43.8
Md.	16	24	8° 02′	0	324	44.3	52.0	53.7	452	40.3	47.0	49.3
Md.	17	24	6° 52′	.042	342	41.6	46.8	49.1	439	40.4	48.2	50.9
N. Y.	18	23	7° 00′	.052	350	41.5	49.5	52.2	375	42.0	47.5	53.9
S. C.	19	18	3° 00′	.030	360	40.9	53.1	59.0	360	34.9	44.7	47.9
Md.	20	[3] 26	7° 40′	.064	371	40.6	46.7	48.9	655	46.3	53.0	56.0
Md.	21	24	4° 35′	.021	377	51.7	58.3	60.2	546	46.9	52.5	53.7
N. Y.	22	24	11° 30′	.080	380	40.5	49.5	52.4	380	39.0	48.0	49.3
N. Y.	23	24	6° 30′	.062	400	42.5	50.5	53.4	400	41.5	51.0	53.7
Md.	24	20	5° 40′	.033	407	43.5	49.4	52.4	550	46.0	52.9	56.2
Minn.[5]	25	19	7° 30′	.062	420	39.2	48.7	50.6	---	---	---	---
N. Y.	26	22	5° 30′	.042	435	37.5	46.5	48.3	460	38.0	47.5	49.9
N. Y.	27	20	6° 00′	.062	440	42.5	55.0	57.3	440	40.0	50.0	50.6
Md.	28	24	4° 35′	.049	450	45.0	52.5	58.5	557	45.5	52.0	53.0
N. Y.[2]	29–*I*	20	4° 00′	.052	460	45.0	55.0	58.9	---	---	---	---
N. Y.[2]	29–*O*	20	2° 45′	.052	---	---	---	---	460	48.0	58.0	60.9
Md.	30	24	5° 40′	.069	469	43.9	49.7	52.1	316	41.5	47.4	49.7
N. Y.	31	20	5° 30′	.042	470	41.0	51.8	53.8	430	40.5	49.0	51.0
N. Y.	32	20	4° 00′	.042	490	39.5	49.5	57.0	500	39.0	49.5	53.1
N. Y.	33	24	3° 30′	.040	500	45.5	54.0	56.4	530	45.0	56.0	58.7
S. C.	34	22	4° 00′	.060	500	40.2	48.7	50.6	500	43.0	52.0	54.9
N. Y.	35	22	4° 30′	.062	510	43.5	53.0	54.6	520	43.0	54.0	57.2

[1] Approach pavement same width as curve except as noted.
[2] Sites 2 and 29 were compound curves, so the curvature of the inside lane differs from that of the outside lane.
[3] Approach pavement 24 feet wide.
[4] Approach pavement 20 feet wide.
[5] At sites 9 and 25 data were not obtained for the outside lane.

8,400 vehicles. Table 1 contains a general description of each location and the observed speed data at the point of minimum sight distance.

Summary of Findings

The analyses of the data included investigations of the coefficient of side friction that vehicles actually develop in traversing horizontal curves, the effect of superelevation on driver behavior, sight distance as related to curvature, speed as related to sight distance and curvature, and actual passenger-car speeds as compared to various standards for safe speeds as based on stopping distances.

The data indicate the following for the conditions of speed and sight distance generally prevailing in the areas where the studies were conducted:

1. Drivers of free-moving passenger cars do not change their speeds appreciably after entering a horizontal curve. Any adjustment in speed that is made because of curvature or limited sight distance is made on the approach to the curve. Observations of vehicle speeds on the approaches were not included in this study.

2. Speeds in the outside lanes were about the same as those in the inside lanes despite the fact that minimum sight distances were, on an average, 20 percent greater in the outside lanes than in the inside lanes of the curves included in these studies. Operating conditions as far as the minimum sight distance is concerned were therefore more critical for the inside lanes than for outside lanes, especially on the sharper curves.

3. The amount of superelevation on the curves studied had no effect on vehicle speeds. For this reason the utilized coefficient of side friction on the same degree of curvature is smaller when the superelevation is high than when it is low. Ten percent of the drivers develop a coefficient of side friction of 0.3 or more on horizontal curves sharper than 15 degrees. A coefficient of side friction of 0.16, however, is rarely exceeded on curves of 6 degrees or less.

4. Superelevation, as normally used in terms of feet of rise per foot of pavement width, without regard to the sharpness of the curve, bears no relation to the percentage of vehicles exceeding the "safe" speed based on curvature, superelevation, and coefficient of side friction. A close correlation exists, however, between *unit superelevation* and the percentage of vehicles exceeding the computed safe speed based on curvature and superelevation; the "unit superelevation" being the feet of rise per foot of width per degree of curvature. The analysis indicates that few vehicles exceed a safe speed on horizontal curves designed with a unit superelevation of more than 0.005 foot per foot of width per degree of curvature. This is a very simple unit to apply in the design of horizontal curves.

5. The minimum sight distance on horizontal curves is not necessarily controlled by or related to the degree of curvature. On the curves studied, however, there was a general tendency for the flatter curves (those of longer radii) to have the longer minimum sight distances.

6. Operating speed and degree of curvature are closely related, and the relation is linear. Drivers do not drive at the high speed permitted by the design on easy curves, and exceed the design speed on sharp curves sometimes by as much as 10 miles per hour.

7. Considering curvature and sight distance only, curvature has a much greater effect on vehicle speed than sight distance.

8. Driver performance on horizontal curves is such that when the minimum sight distance is 400 feet or longer, few drivers exceed what can be considered a safe speed regardless of which of the commonly employed factors is used in computing driver stopping distances. With the shorter sight distances, however, most drivers stay within a speed from which they could come to a stop within the available sight distance only if no allowance is made for perception and reaction time. Between these two extremes the percentage of vehicles exceeding the safe speed depends on the criterion used to determine the safe speed.

From these studies it appears that sight distances should be at least 400 feet (if measured from a height of 4½ feet to 4 inches) on horizontal curves on two-lane main rural highways if drivers are to be expected to stop when an object suddenly appears in their lane.

9. Vehicle speeds are considerably lower on horizontal curves than on vertical curves with the same minimum sight distance.

New York Studies

In the New York studies, vehicle speeds were recorded in each of the two lanes at 100-foot intervals for a distance of 1,000 feet including the sharpest sections of the horizontal curves. The individual car speeds were obtained by measuring the time it took vehicles to travel the 100-foot distances between the stations on each curve. A distribution of vehicle speed and the average speed were thus obtained for vehicles while in each of the ten 100-foot sections. Using sight distances recorded separately for each direction of travel, the design speed based on the American Association of State Highway Officials standards for nonpassing sight distances only was determined for each curve in each direction. The percentage of cars exceeding the design speed determined in this manner at each 100-foot station was then obtained from the speed distribution.

Typical data recorded in New York are plotted for one location in the chart at the left in figure 1. The bottom portion of the chart shows the sight distances on the curve for

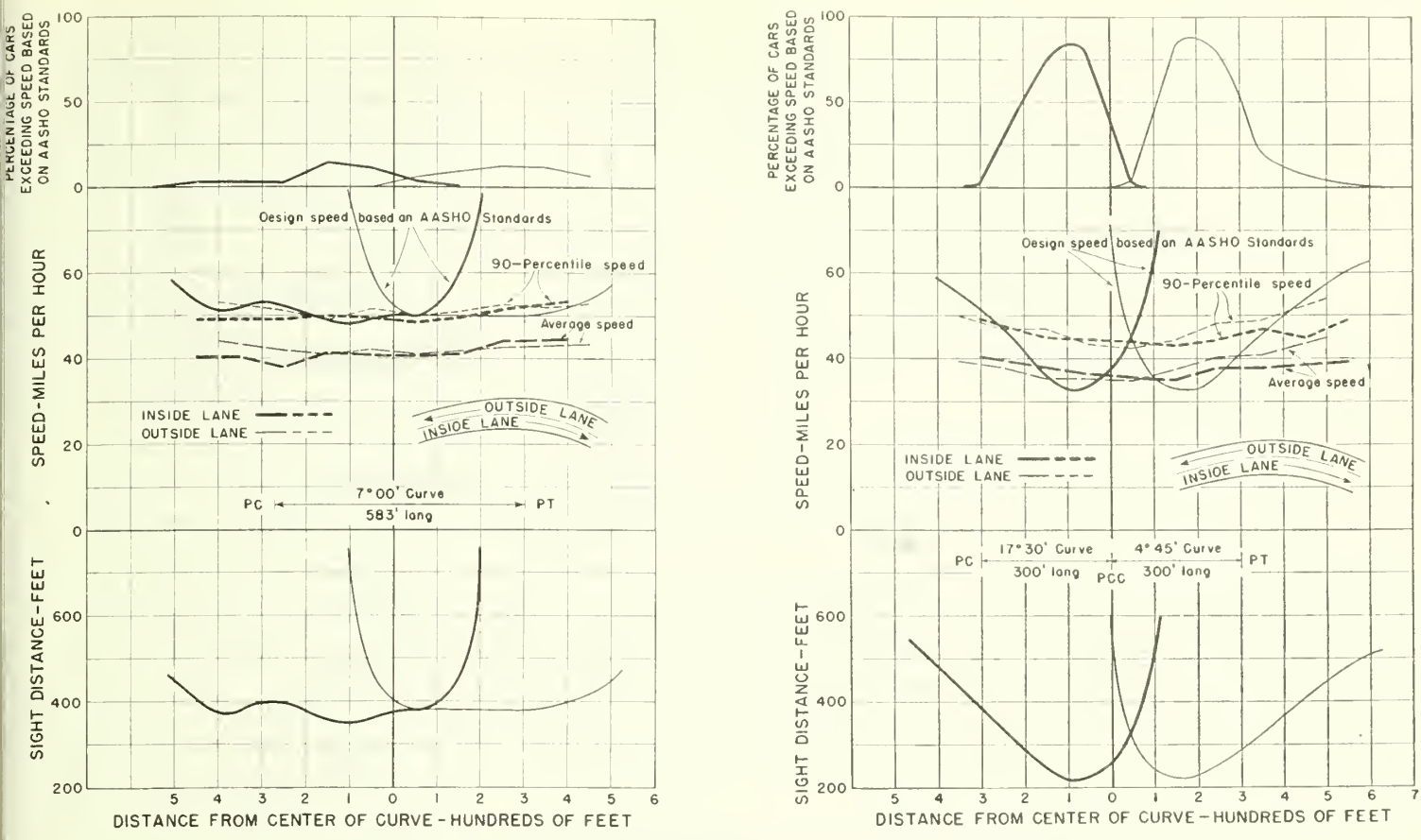

Figure 1.—Typical data recorded at two locations in New York State.

each direction of travel. The middle portion shows the average speeds, the 90-percentile speeds, and the design speeds based on A.A.S.H.O. nonpassing sight-distance standards. (The 90-percentile speed is that speed exceeded by only 10 percent of the vehicles.) As indicated, this was a 7° 00′ curve, 583 feet long. The upper portion of the chart shows the percentage of vehicles exceeding the design speed at the several points on the curve.

The minimum sight distance for each direction of travel occurs about 100 feet ahead of the center of the curve, and this is characteristic of all the horizontal curves studied. It will be noted that for the inside lane, where the minimum sight distance is 350 feet, the average and the 90-percentile free-moving passenger-car speeds were 41 and 50 miles per hour, respectively.

The chart at the right in figure 1 is for a location where the minimum sight distance was only about 200 feet. The two charts illustrate an important driver characteristic observed at all of the locations in New York—drivers of free-moving passenger cars do not change their speeds appreciably after entering a horizontal curve, even when the curvature is as sharp as 15 degrees. Most of the adjustment in speed that is made, whether because of curvature, limited sight distance, or other reasons, is made on the approach to the curve. (Observations of changes in speed on the approaches were not included in this study.) Furthermore, a rather high percentage of drivers travel around curves with short sight distances at speeds which are higher than those based on the A.A.S.H.O. design standards for nonpassing sight distance.

Even though drivers do not materially reduce their speeds while on a curve, there is a definite relation between operating speeds and the minimum sight distances on different curves in New York. Figure 2 shows two sets of speed distributions for horizontal curves having minimum sight distances between 200 and 550 feet, by 50-foot increments. (No locations were studied where the minimum sight distance was between 250 and 299 feet.) The curves on the left show the distributions

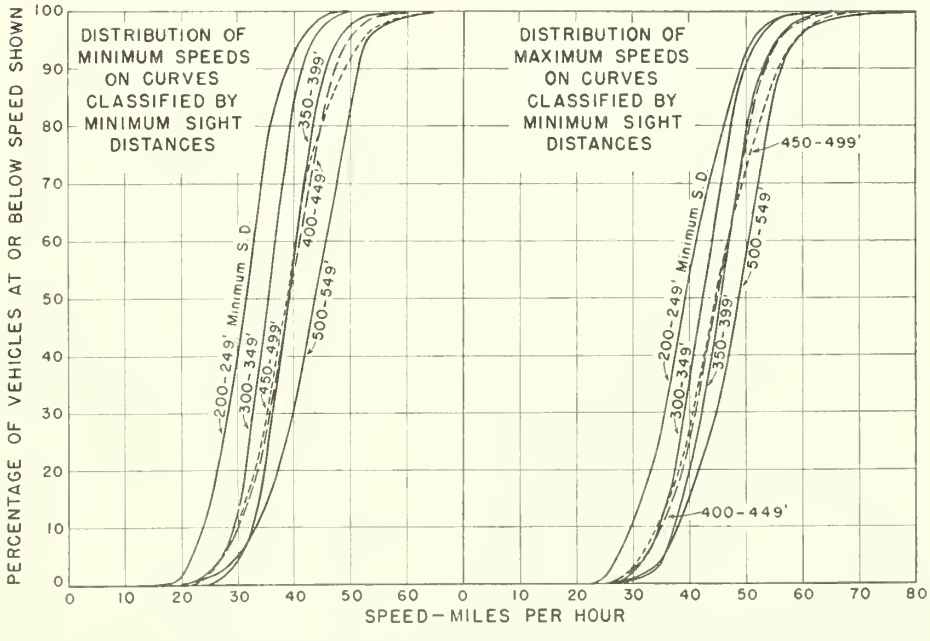
DISTRIBUTION OF MINIMUM SPEEDS ON CURVES CLASSIFIED BY MINIMUM SIGHT DISTANCES
DISTRIBUTION OF MAXIMUM SPEEDS ON CURVES CLASSIFIED BY MINIMUM SIGHT DISTANCES
PERCENTAGE OF VEHICLES AT OR BELOW SPEED SHOWN
200-249' Minimum S.D
300-349'
350-399'
400-449'
450-499'
500-549'
SPEED-MILES PER HOUR

Figure 2.—Distribution of free-moving passenger-car speeds on horizontal curves of two-lane highways in New York State.

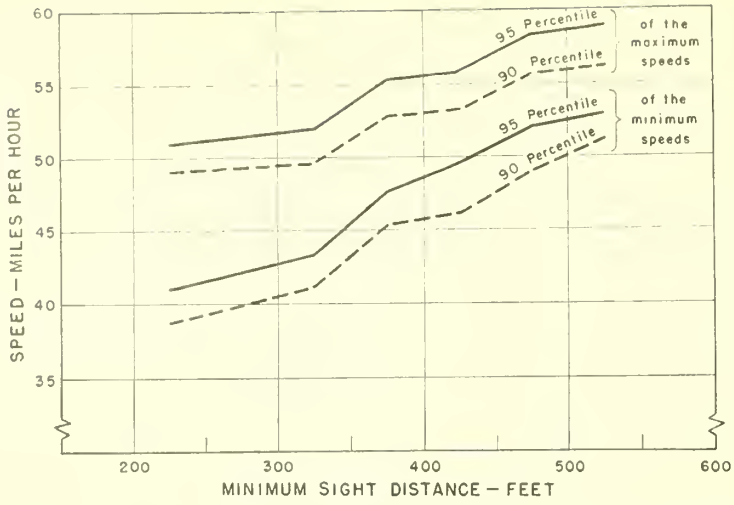

Figure 3.—Relation of 90- and 95-percentile speeds to minimum sight distance.

of the minimum speeds for the various sight-distance conditions. For this purpose the minimum speed of each driver, regardless of where the minimum speed occurred on the curve, was used to determine the frequency distribution. Similarly, the set of curves on the right shows distributions of maximum speeds for the several horizontal curves falling in the same sight-distance groupings.

It will be noted that for both minimum and maximum speeds, the speeds are considerably lower at locations where the minimum sight distance is below 250 feet than at locations where the minimum sight distance is greater than 500 feet. There is little difference between the speed distributions for the three minimum-sight-distance groups that range from 350 to 500 feet. There is a consistent relation between the speeds of the faster drivers (90-percentile speed or higher) and the minimum sight distance.

This relation between the speed and minimum sight distance for the higher speed drivers is shown in a different form in figure 3. It may be noted that for each of the four curves the speeds increase progressively as the minimum sight distances increase, although not at a uniform rate. Speeds of the faster group of drivers are related in some degree to the available sight distance on horizontal curves. This might appear to contradict the data presented in figure 1, but further examination will reveal that this is not the case. Figure 1 shows the average behavior of all drivers at each 100-foot station around a horizontal curve and, as stated before, the speed of the average driver does not reflect the rapid change in the sight distance at different points on the curve. In figures 2 and 3, however, the minimum and maximum speeds of individual drivers have been related to the minimum sight distances for the different horizontal curves. The minimum and the maximum speeds of different drivers do not necessarily occur at the same points on a horizontal curve. The variations in the make and model of car, in the individual perception and reaction times and other driver characteristics, undoubtedly affect the individual reactions to highway conditions.

The foregoing discussion was confined to the results of the studies conducted in New York because only in these studies were the vehicle speeds observed over a length of highway. In the remainder of this article, relations between speed and sight distance and other factors involve only the speeds at the points of minimum sight distance, and accordingly data for all locations studied are included.

Sight Distance–Curvature Relation

Drivers approaching horizontal curves can see that the sight distances are shorter on some curves than on others, and to some extent they can judge that one curve is sharper than another. Undoubtedly, drivers control their speed on the basis of their experience with respect to one or both of these factors.

Under some conditions, as in a cut or in a wooded area, the minimum sight distance on horizontal curves would vary inversely with the degree of curvature, but under the more usual conditions, where sight distance is determined by other than cross section, this is seldom the case. Among the locations studied for this report there are some very sharp curves where the minimum sight distance is greater than at some of the flatter curves.

Figure 4 shows the relation between minimum sight distance and degree of curvature as found on the 35 curves studied. The points are widely scattered. In extreme, at locations where the curvatures are about 5 degrees the sight distances range from 220 to 560 feet; conversely, the curvature ranges from 4 to 29 degrees for the curves with 300-foot minimum sight distance.

Mathematical analysis employing the method of least squares was applied to the basic data to determine the interrelation between minimum sight distance and curvature. The analysis showed that the minimum sight distance on these horizontal curves is not necessarily controlled by or related to the degree of curvature. There is, however, a general tendency, as should be expected, for the flatter curves to have the greater minimum sight distances. On the average, for each 100-foot change in sight distance, there appears to be a change of about 3 degrees in curvature.

Even though, in open and level terrain, the available sight distance is not necessarily related to the curvature, the design of highway curvature and superelevation is based on the probable future speeds and an effort is made to provide adequate sight distance for safe operation at those speeds. It is important, therefore, to determine to what extent drivers are influenced in their speeds by the combina-

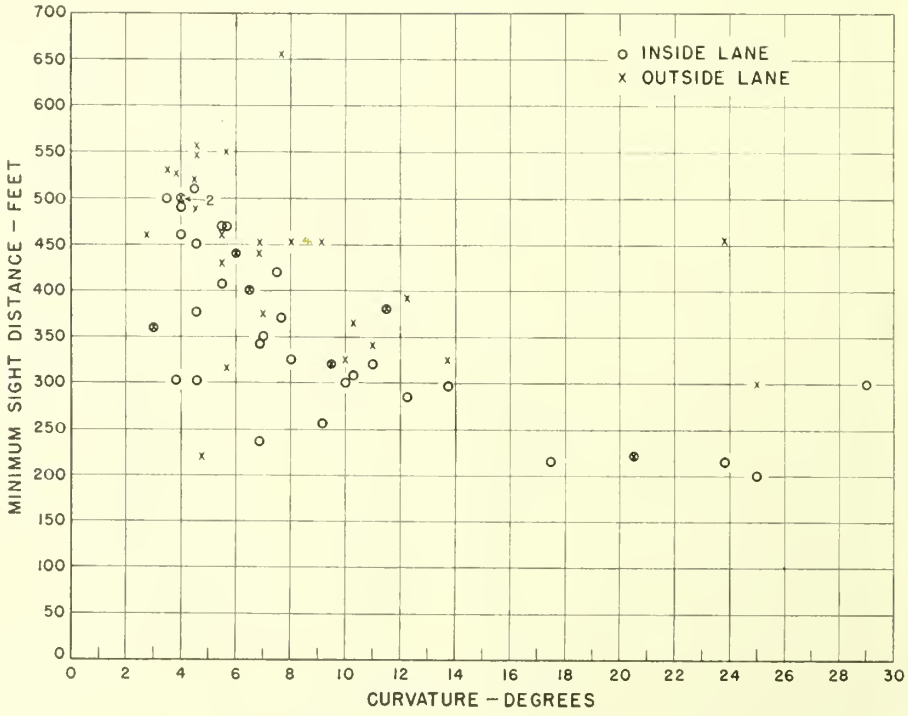

Figure 4.—Relation between minimum sight distance and curvature.

tion of curvature and superelevation and by the available sight distances.

Speed–Curvature Relation

The speeds at which drivers operated on the curves of various degrees are shown in figure 5. The points for these curves were plotted by combining the inside and outside lane speed data shown in table 3. For this purpose it was found not to be necessary to plot the lane speeds separately because for the same curve the average difference was only 0.2 mile per hour. (The arrangement within groups in table 3 was governed by considerations which will be explained later.)

Points are shown only for the average speed for each of the curvature groups in the table. The point shown for a curvature of 16 to 20 degrees represents only 1 location, but all the other points represent data averaged for 7 to 20 locations. The points for the 90- and 95-percentile curves are not shown, but they fell as close to their respective curves as those for the average speeds. Points representing speeds at the individual locations as well as those for the groups also came remarkably close to the curves.

The method of least squares was used to fit a straight line, a hyperbola, and a parabola to the data for the individual locations, and the straight-line relation between speed and curvature was found to give the best fit. The resulting equations, with the corresponding standard errors and coefficients of correlations, are shown in table 2. The high coefficients of correlation found for these equations indicate that operating speeds are very closely related to the degree of curvature for the range between 2-degree and 30-degree curves included in this study. The average speed is lowered by 3 miles per hour for each 4 degrees that the curvature increases, and the 95-percentile speed is lowered by 1 mile per hour for each 1-degree increase in curvature.

Included in figure 5 is a curve showing speeds that are presumed to be safe for the various degrees of curvature. These "safe" speeds are based on the average superelevation for each of the curvature groups in table 3 and the current standards of highway design [2] as shown in figure 6. This is a curved relation between speed and curvature, whereas the actual performance of drivers is a straight-line relation. A driver traveling at the average speed of all free-moving vehicles does not exceed what is considered a safe speed on any of the curves. The fastest 10 percent of the drivers, however, do exceed the safe speeds on curves sharper than 8 degrees, and on curves sharper than 16 degrees the average driver travels at about the safe speed, indicating that nearly half of the drivers exceed the safe speed.

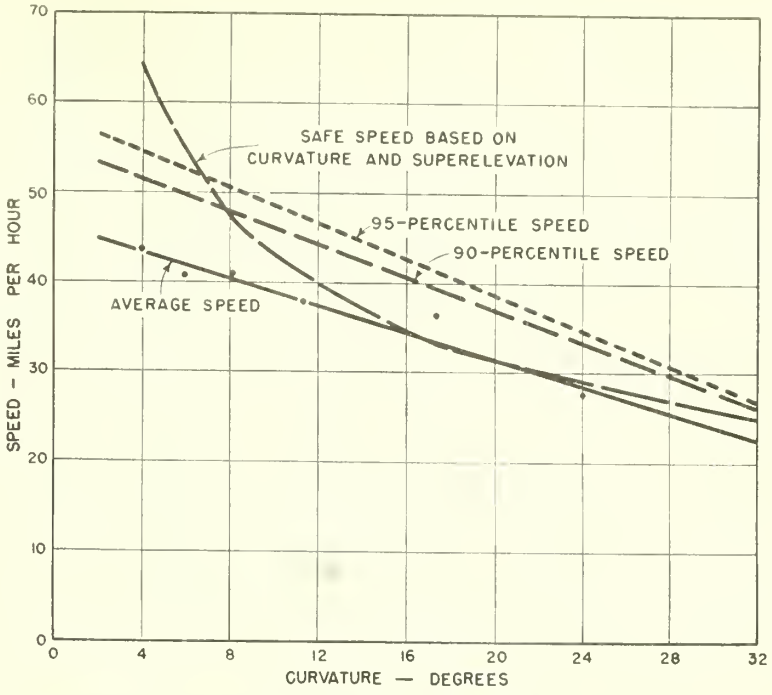

Figure 5.—Relation between speed and horizontal curvature.

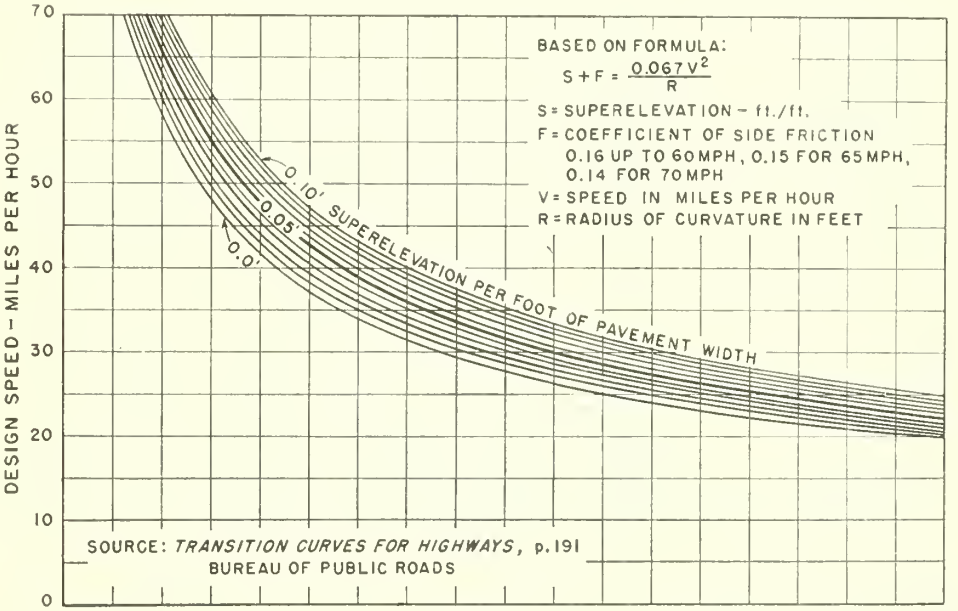

Figure 6.—Maximum curvature for various assumed design speeds.

[2] *Transition curves for highways*, by Joseph Barnett. Bureau of Public Roads, 1940. p. 191.

Table 2.—Relation between speed and horizontal curvature

Speed	Equation [1]	Standard error (adjusted)	Coefficient of correlation (adjusted)
		M.p.h.	
Average	$V_a = 46.26 - 0.746\ D$	3.15	0.819
90-percentile	$V_{90} = 55.22 - 0.909\ D$	3.29	.858
95-percentile	$V_{95} = 58.46 - 1.000\ D$	3.51	.863

[1] V=speed in miles per hour; D=curvature in degrees.

At individual locations included in this study, 10 percent of the drivers exceeded the safe speed by as much as 10 miles per hour. It is apparent, therefore, that when the road is clear and dry many drivers actually utilize a coefficient of side friction which exceeds that intended in modern highway design. Normally a low value of side friction is purposely used in design to provide some margin of safety. To reduce the needed side friction, highway designers make use of superelevation. Generally, however, the superelevation is limited to a maximum of 0.10 foot per foot, and even on the sharpest curves included in

Table 3.—Coefficient of side friction as related to speed on horizontal curves on two-lane highways

Group and site No.[1]	Curvature group	Superelevation	Inside lane of curve							Outside lane of curve						
			Minimum sight distance	Speed at minimum sight distance			Coefficient of side friction			Minimum sight distance	Speed at minimum sight distance			Coefficient of side friction		
				Average	90-percentile	95-percentile	Average	90-percentile	95-percentile		Average	90-percentile	95-percentile	Average	90-percentile	95-percentile
		Ft./ft.	*Feet*	*M.p.h.*	*M.p.h.*	*M.p.h.*				*Feet*	*M.p.h.*	*M.p.h.*	*M.p.h.*			
1A:																
11	3°50′	0.014	303	46.6	55.1	58.5	0.083	0.122	0.139	526	46.3	53.0	55.5	0.082	0.112	0.124
21	4°35′	.021	377	51.7	58.3	60.2	.122	.161	.173	546	46.9	52.5	53.7	.097	.127	.133
12	4°35′	.028	303	40.8	46.5	49.0	.061	.088	.101	489	42.4	47.2	50.4	.068	.091	.108
19	3°00′	.030	360	40.9	53.1	59.0	.029	.069	.092	360	34.9	44.7	47.9	.013	.040	.050
33	3°30′	.040	500	45.5	54.0	56.4	.043	.077	.088	530	45.0	56.0	58.7	.041	.086	.099
Average	3°54′	.027	369	45.1	53.4	56.6	.066	.103	.119	490	43.1	50.7	53.2	.058	.090	.102
1B:																
32	4°00′	.042	490	39.5	49.5	57.0	.031	.073	.110	500	39.0	49.5	53.1	.029	.073	.090
28	4°35′	.049	450	45.0	52.5	58.5	.059	.099	.134	557	45.5	52.0	53.0	.062	.096	.101
29–*I*	4°00′	.052	460	45.0	55.0	58.9	.043	.090	.110							
29–*O*	2°45′	.052								460	48.0	58.0	60.9	.022	.056	.067
34	4°00′	.060	500	40.2	48.7	50.6	.016	.051	.060	500	43.0	52.0	54.9	.016	.066	.081
35	4°30′	.062	510	43.5	53.0	54.6	.038	.086	.095	520	43.0	54.0	57.2	.035	.091	.110
I–Average	4°13′	.053	482	42.6	51.7	55.9	.037	.079	.101							
O–average	3°58′	.053								507	43.7	53.1	55.8	.036	.078	.091
2A:																
24	5°40′	.033	407	43.5	49.4	52.4	.093	.129	.149	550	46.0	52.9	56.2	.107	.153	.177
5	6°52′	.036	236	38.6	46.3	47.6	.084	.136	.146	452	36.9	42.1	43.9	.073	.106	.119
31	5°30′	.042	470	41.0	51.8	53.8	.066	.130	.144	430	40.5	49.0	51.0	.063	.112	.125
26	5°30′	.042	435	37.5	46.5	48.3	.048	.097	.108	460	38.0	47.5	49.9	.051	.103	.118
17	6°52′	.042	342	41.6	46.8	49.1	.097	.134	.152	439	40.4	48.2	50.9	.089	.114	.166
Average	6°05′	.039	378	40.4	48.1	50.2	.077	.125	.140	466	40.4	47.9	50.4	.077	.124	.141
2B:																
27	6°00′	.062	440	42.5	55.0	57.3	.065	.150	.168	440	40.0	50.0	50.6	.050	.113	.118
23	6°30′	.062	400	42.5	50.5	53.4	.075	.132	.155	400	41.5	51.0	53.7	.069	.136	.157
2–*O*	4°45′	.062								220	38.0	43.0	46.5	.018	.041	.058
30	5°40′	.069	469	43.9	49.7	52.1	.059	.095	.111	316	41.5	47.4	49.7	.045	.080	.095
I–average	6°03′	.064	436	43.0	51.7	54.3	.067	.125	.145							
O–average	5°35′	.063								344	40.2	47.8	50.1	.042	.086	.101
3A:																
16	8°02′	0	324	44.3	52.0	53.7	.184	.254	.271	452	40.3	47.0	49.3	.153	.208	.228
6	9°10′	.042	256	41.4	47.5	49.1	.141	.200	.216	453	40.7	47.1	49.5	.136	.196	.221
18	7°00′	.052	350	41.5	49.5	52.2	.089	.149	.171	375	42.0	47.5	53.9	.092	.133	.186
Average	8°04′	.031	310	12.4	49.7	51.7	.139	.202	.221	427	41.0	47.2	50.9	.128	.179	.213
3B:																
25–*I*	7°30′	.062	420	39.2	48.7	50.6	.072	.146	.162							
20	7°40′	.064	371	40.6	46.7	48.9	.084	.132	.150	655	46.3	53.0	56.0	.128	.188	.217
14	9°30′	.073	320	37.8	45.0	47.6	.086	.152	.179	320	37.0	44.2	46.3	.079	.144	.165
I–average	8°13′	.066	370	39.2	46.8	49.0	.081	.141	.165							
O–average	8°35′	.068								487	41.6	48.6	51.2	.106	.169	.195
4A:																
13	10°18′	.038	308	41.9	46.3	47.1	.174	.220	.229	377	37.0	44.2	46.5	.127	.197	.222
7	12°15′	.045	285	35.7	41.5	42.9	.137	.201	.218	391	35.2	40.4	42.9	.132	.189	.218
8	13°44′	.068	297	40.4	49.7	52.9	.194	.329	.381	323	41.2	46.9	48.6	.205	.285	.311
Average	12°06′	.050	297	39.3	45.8	47.6	.166	.248	.271	364	37.8	43.8	46.0	.152	.221	.249
4B:																
10	10°00′	.073	300	35.5	44.0	46.4	.074	.153	.179	325	37.5	44.3	45.5	.091	.156	.169
15	11°00′	.073	320	36.0	45.0	46.8	.094	.187	.208	340	35.5	42.0	43.8	.085	.154	.174
22	11°30′	.080	380	40.5	49.5	52.4	.140	.250	.289	380	39.0	48.0	49.3	.124	.230	.247
Average	10°50′	.075	333	37.3	46.2	48.5	.101	.195	.223	348	37.2	44.8	46.2	.100	.179	.196
5:																
2–*I*	17°30′	.062	215	36.5	44.0	45.0	.211	.335	.352							
6A:																
9–*I*	29°00′	.062	300	25.1	29.7	31.5	.151	.236	.274							
3	23°50′	.077	215	31.5	36.2	38.1	.200	.289	.328	455	32.2	36.2	37.8	.212	.289	.319
I–average	26°25′	.070	258	28.3	33.0	34.8	.177	.266	.304							
O–average	23°50′	.077								455	32.2	36.2	37.8	.212	.289	.319
6B:																
1	25°00′	.083	200	23.7	28.9	29.4	.081	.161	.170	300	22.6	28.5	29.4	.066	.155	.170
4	20°30′	.083	220	28.0	34.5	36.0	.103	.202	.227	220	30.0	35.0	37.2	.132	.210	.248
Average	22°45′	.083	210	25.8	31.7	32.7	.094	.184	.201	260	26.3	31.8	33.3	.101	.186	.212

[1] The outside and inside lanes have different curvatures at sites 2 and 29, and no outside lane data were obtained at sites 9 and 25. Because of these cases (as indicated by *I* for inside lane and *O* for outside lane), separate group averages for inside and outside lanes are given for some groups.

this study the maximum was only 0.083 foot per foot.

The family of curves in figure 6, used to represent current design practices, is based on a safe coefficient of side friction of 0.16 for speeds up to 60 miles per hour, 0.15 for 65 miles per hour, and 0.14 for 70 miles per hour.

Side Friction Utilized

One of the factors of highway design for which factual data have been seriously lacking is the coefficient of side friction that vehicles actually develop as they negotiate various curves. The coefficients of side friction developed on the horizontal curves included in this study were determined from the data recorded, using the following basic formula:

$$F=\frac{0.067V^2}{R}-S$$ where

F = coefficient of side friction.
V = speed in miles per hour.
R = radius of curve in feet.
S = superelevation in feet per foot.

The basic data for each of the curves included in the study and the calculated coefficients of side friction are shown in table 3. In this table the horizontal curves have first been arranged in groups according to degree of curvature. Each of these groups has been further divided into two subgroups, the first subgroup including curves having relatively low superelevations and the other subgroup including curves having the higher superelevations.

The superelevations used to separate the data into the subgroups were related to the curvatures. For 3- and 4-degree curves the division was made at a superelevation of about 0.04 foot per foot. This value increases as the degree of curvature increases, and for the sharper curves the division was made at about 0.08 foot per foot. The coefficients of side friction are shown for the average speed, for the 90-percentile speed, and for the 95-percentile speed on each curve.

Figure 7, showing separately for inside and outside lanes the relation of speed, coefficient of side friction, and superelevation, was plotted from the average values for each group of curves in table 3. The lower chart of each pair shows the average, 90-, and 95-percentile speeds for each curvature group; the upper chart shows the corresponding coefficient of side friction utilized in negotiating the various curves at the indicated speeds.

It may be noted from figure 7 that the operating speeds are about the same on horizontal curves of similar degree, regardless of the superelevations, within the limits of this study. This appears to be true for both the inside and outside lanes. The amount of super

elevation, therefore, apparently had little or no effect on the operating speeds.

It follows, then, that the utilized coefficient of side friction was smaller when the superelevation was high than when it was low. For example, for the inside lanes (figure 7) on curves of 10 to 15 degrees the side friction for drivers traveling at the average speed was 0.17 when the superelevation was 0.050 foot per foot and 0.10 when the superelevation was 0.075 foot per foot. The high-speed drivers developed higher coefficients of side friction: At the 95-percentile speed, coefficients of side friction averaged 0.28 when the superelevation was 0.50 and 0.22 when the superelevation was 0.075 foot per foot. In both of these cases the difference in side friction between the low and higher superelevations was about the same, being 0.06 in the one example and 0.07 in the other. On curves of less than 7 degrees the difference was less, being in the neighborhood of 0.02 or 0.03.

Figure 7 also shows that the utilized coefficient of side friction generally increased as the degree of curvature increased, and that the coefficient was slightly smaller for the outside than for the inside lanes.

Critical Criterion of Superelevation

Since it was found that curvature affected the operating speeds on horizontal curves but that superelevation had little or no effect, the analysis was directed to the percentage of vehicles that exceeded safe speeds based on curvature and superelevation. Superelevation is normally expressed in units of feet of rise per foot of pavement width, and curvature in degrees. To facilitate the determination of the number of vehicles that were operating at unsafe speeds in relation to the geometric features of the highway, it was found desirable to express the superelevation in terms of feet of rise per foot of pavement width per degree of curvature, a term hereafter identified as the *unit of superelevation.*

An extremely high degree of correlation was found to exist when this unit of superelevation was related to the percentage of vehicles exceeding the safe speed, as shown in figure 8 Only one curve is shown for both lanes of travel, because the individual curves for the inside and outside lanes coincide. This is easily understood if it is remembered that the curvature was measured to the center of the highway, not separately for each lane, and that operating speeds were nearly the same in both directions of travel. The actual plotting points are not shown in figure 8, but they were all very close to the average curve, indicating a very consistent relation between the two variables. This was the case even though both sharp and moderate curves were included over the entire range of unit superelevations used.

The curve breaks very sharply at a unit

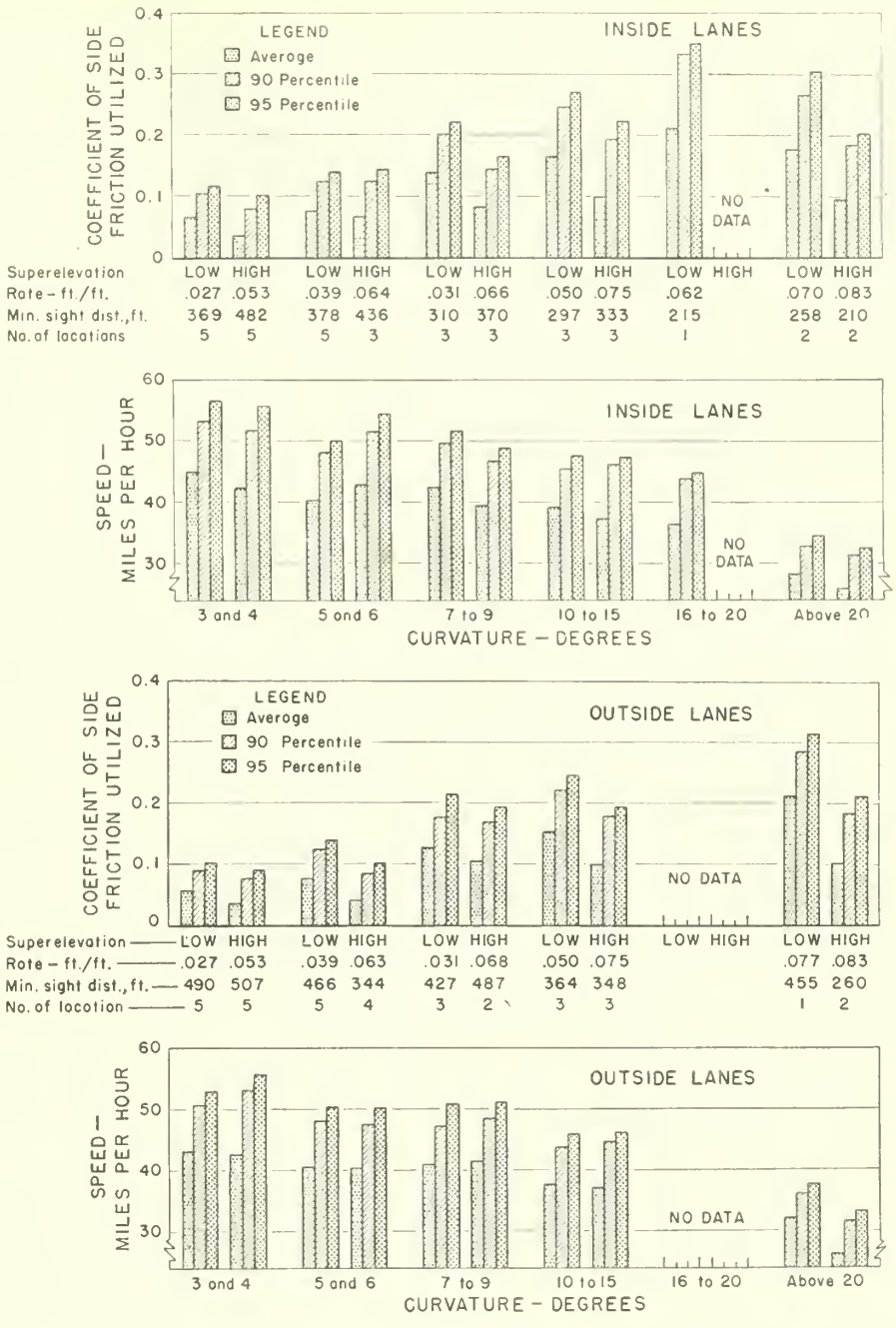
COEFFICIENT OF SIDE FRICTION UTILIZED
LEGEND
Average
90 Percentile
95 Percentile
INSIDE LANES
NO DATA
Superelevation LOW HIGH LOW HIGH LOW HIGH LOW HIGH LOW HIGH LOW HIGH
Rate – ft./ft. .027 .053 .039 .064 .031 .066 .050 .075 .062 .070 .083
Min. sight dist., ft. 369 482 378 436 310 370 297 333 215 258 210
No. of locations 5 5 5 3 3 3 3 3 1 2 2
SPEED – MILES PER HOUR
INSIDE LANES
NO DATA
3 and 4
5 and 6
7 to 9
10 to 15
16 to 20
Above 20
CURVATURE – DEGREES
COEFFICIENT OF SIDE FRICTION UTILIZED
LEGEND
Average
90 Percentile
95 Percentile
OUTSIDE LANES
NO DATA
Superelevation LOW HIGH LOW HIGH LOW HIGH LOW HIGH LOW HIGH LOW HIGH
Rate – ft./ft. .027 .053 .039 .063 .031 .068 .050 .075 .077 .083
Min. sight dist., ft. 490 507 466 344 427 487 364 348 455 260
No. of location 5 5 5 4 3 2 3 3 1 2
SPEED – MILES PER HOUR
OUTSIDE LANES
NO DATA
3 and 4
5 and 6
7 to 9
10 to 15
16 to 20
Above 20
CURVATURE – DEGREES

Figure 7.—(Above) Relation between speed, coefficient of side friction, and superelevation

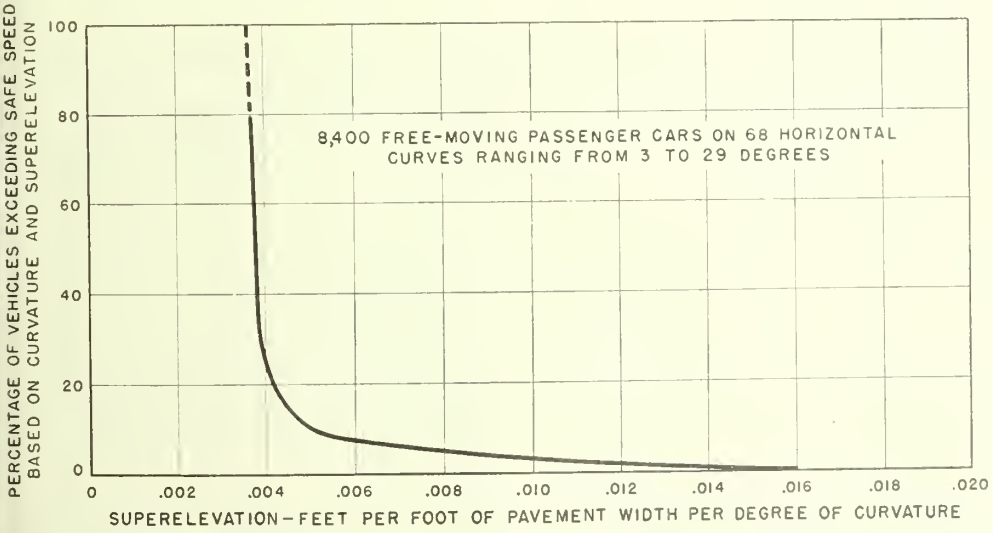
PERCENTAGE OF VEHICLES EXCEEDING SAFE SPEED BASED ON CURVATURE AND SUPERELEVATION
8,400 FREE-MOVING PASSENGER CARS ON 68 HORIZONTAL CURVES RANGING FROM 3 TO 29 DEGREES
SUPERELEVATION – FEET PER FOOT OF PAVEMENT WIDTH PER DEGREE OF CURVATURE

Figure 8.—Effect of superelevation on percentage of vehicles exceeding safe speed.

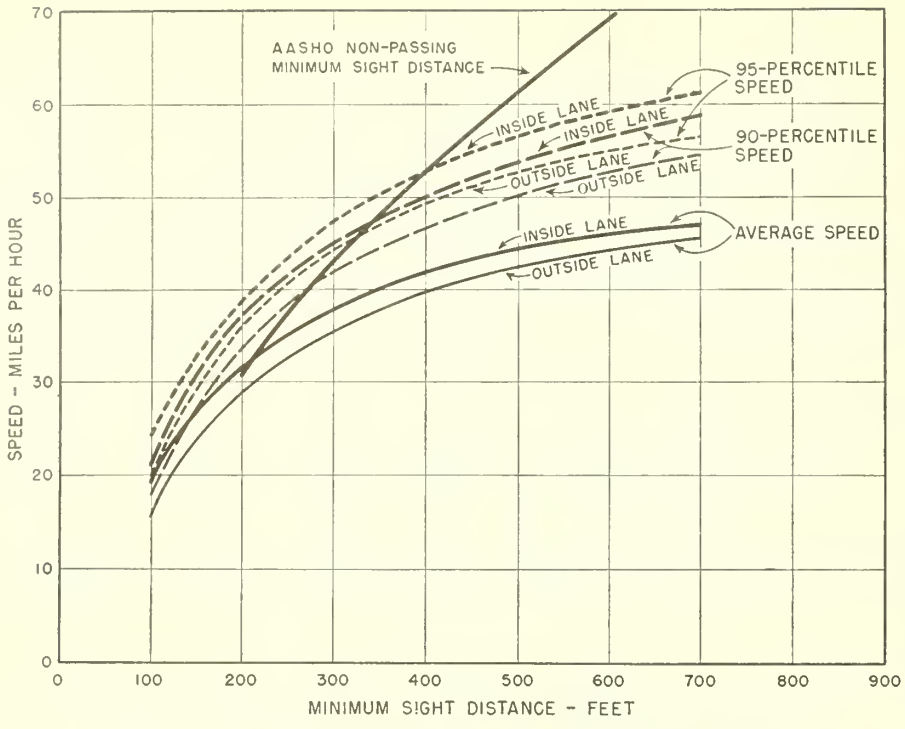

Figure 9.—Relation between speed and minimum sight distance.

superelevation of about 0.005. This indicates that for conditions included in this study few vehicles exceed a safe speed on horizontal curves designed with a unit superelevation of more than 0.005 foot per foot of pavement per degree of curvature. For unit superelevations less than this amount, however, a high percentage of the vehicles will exceed the speeds considered safe for the combination of curvature, superelevation, and coefficients of side friction as shown in figure 6. Although there are curves with unit superelevations of less than 0.005 foot per foot per degree, specifications in modern design standards generally provide higher rates.

Speed–Minimum Sight Distance Relation

In addition to consideration of curvature and superelevation, modern design practices require sight distances adequate for safe operation. This study provides data for an analysis of the extent to which sight distance on horizontal curves affects driver speeds. The relation between speed and sight distance, neglecting the effect of curvature, is shown in figure 9, the curves being based on the data in table 1.

One fact clearly brought out in the figure is that, at a given minimum sight distance, vehicle speeds are higher on the inside lanes than on the outside lanes. At a given location the observed speeds were about the same in both lanes but, because the sight distances are longer for a driver traveling in the outside lane than in the inside lane, the relation shown in figure 9 is obtained. On an average, the minimum sight distance in the outside lane was about 20 percent higher than in the inside lane. Taking this into consideration, it appears that the drivers in the inside lanes are more apt to exceed safe speed with respect to sight-distance requirements than when they are traveling in the outside lanes.

As previously stated, the relation between available sight distance and speed as shown in figure 9 entirely ignores the fact that at the locations with the shorter sight distances the curvatures are generally sharper than at the locations with the longer sight distances. The change in speed with a change in sight distance as shown in figure 9 may therefore have been caused largely by the driver's reaction to curvature rather than sight distance.

As may be observed from figure 9, the operating speeds are related to the minimum sight distance, but the data at the individual locations vary considerably about the established theoretical relations shown in table 4. This is indicated by the relatively high standard errors, which are an index of the dispersion of the individual points about the curves. These standard errors are much greater than those for speed as related to curvature which are shown in table 2. This was the case even though every effort was made to determine whether there was not a more direct relation between speed and sight distance. The hyperbolic equations shown in table 4 were found by the method of least squares to give the best fit of the individual points to the curves. Other types of general equations tried were the straight line, parabola, and two additional hyperbolas.

Although the speeds of drivers on horizontal curves are not principally governed by sight-distance conditions, it is of interest to examine the extent to which drivers exceed safe speeds as determined by various sight-distance criteria.

Included in figure 9 is a curve showing the A.A.S.H.O. recommended nonpassing minimum sight distance for given speeds. The average driver was found to operate his car at a speed from which he could have stopped safely, according to these standards, on all horizontal curves for which the minimum sight distance was about 200 feet or more. The high-speed (95-percentile) drivers operated their cars at speeds which would permit them to stop within the available sight distance only when sight distances were above 400 feet.

Since drivers do not control their speeds to conform with the available sight distances as related to total stopping distances, it is of interest to determine what portion of the total stopping distances, for the speeds actually observed, is provided by the available sight distance. Results from several studies show the distances necessary to bring vehicles to a stop from various speeds. Among other things, these tests show the braking distance, which is the distance required to stop from the moment the brakes are applied until the car comes to a standstill. Not so readily determinable is the driver (or total) stopping distance, which is the braking distance plus the distance the car travels during the driver perception and reaction time. Table 5 shows the braking and driver stopping distances

Table 4.—Theoretical equations showing relation between speed and minimum sight distance

Speed	Equation [1]	Standard error (adjusted)	Coefficient of correlation (adjusted)
Inside lanes:		*M.p.h.*	
Average	$V_a=56.8-\frac{75.4}{S+1}$	4.55	0.613
90-percentile	$V_{90}=69.5-\frac{97.0}{S+1}$	5.14	.643
95-percentile	$V_{95}=74.6-\frac{108.8}{S+1}$	5.24	.708
Outside lanes:			
Average	$V_a=55.6-\frac{80.2}{S+1}$	4.14	.623
90-percentile	$V_{90}=66.1-\frac{96.3}{S+1}$	4.64	.657
95-percentile	$V_{95}=68.8-\frac{97.7}{S+1}$	5.14	.615

[1] V=speed in miles per hour; S=minimum sight distance in hundreds of feet.

Table 5.—Relation of braking and driver stopping distances to speed

Speed	Braking distance from study [1]			Driver stopping distance [2]			
				From study [3]		A.A.S.H.O. (No safety factor)	A.A.S.H.O. (1.25 safety factor)
	15-percentile	Average	85-percentile	15-percentile	85-percentile		
M.p.h.	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
20	16	22	29	110	123	115	120
30	36	45	63	168	195	180	192
40	64	80	118	226	280	252	275
50	100	134	210	283	393	331	368
60	144	205	336	342	534	427	484
70	196	310	510	402	716	533	614

[1] *Braking distances of vehicles from high speeds and tests of friction coefficients*, by O. K. Normann. PUBLIC ROADS, vol. 7, No. 8, June 1953.
[2] Includes driver perception and reaction distance.
[3] Perception and reaction distance same as in A.A.S.H.O. policy.

According to the A.A.S.H.O. recommended design standards for rural highways and the results of recent braking distance tests.

The 15-percentile, average, and 85-percentile braking distances are those recorded during a comprehensive study of braking distances of vehicles.[3] These percentiles were used, rather than the 90- and 95-percentiles used elsewhere in this article, because they were the 2 extremes that could be obtained with reasonable accuracy from the braking-distance information. The 15-percentile braking distance is the distance within which 15 percent of the vehicles with the best brakes could stop, and the 85-percentile braking distance is the distance within which 85 percent of the vehicles could stop, with only 15 percent requiring longer distances.

Figure 10 shows graphically the data contained in table 5. The individual curves are each identified by a number corresponding to the appropriate column in table 5. Figure 10 shows, for example, that for speeds above 60 miles per hour the distance within which 85 percent of the vehicles will stop after the brakes are applied (curve 4) is greater than the driver (total) stopping distance for the 15 percent of the drivers that made the shortest stops (curve 5).

The various relations between speed and distance under the several conditions represented in figure 10 were used to relate driving speeds on horizontal curves to sight-distance conditions. The object was to use each of the several curves in figure 10 as a criterion to obtain a measure of the relative ability of drivers to stop their vehicles within the available sight distances from the speeds at which they were traveling on the various horizontal curves.

The percentages of vehicles traveling on horizontal curves at speeds exceeding those obtained from the several assumed criteria are shown in tables 6 and 7 for the inside and outside lanes, respectively. Table 6 shows, for example, that at site 1 the minimum sight distance was 200 feet. Curve 6 in figure 10 shows that 85 percent of the drivers during the braking distance tests could stop within 200 feet from a speed of 30.6 miles per hour. This speed is recorded in the third column of table 6. The seventh column shows that none of the drivers observed on the curve at this site exceeded a speed of 30.6 miles per hour.

The data in tables 6 and 7 were plotted as smooth curves in figure 11, which shows the percentages of vehicles exceeding the speeds from which drivers can brake or stop within the available sight distances on the horizontal curves. Four pairs of curves are shown, each pair consisting of information for the inside lane and for the outside lane.

At locations where the minimum sight distance is 400 feet or longer, few drivers exceed the speed from which they are able to stop within the available sight distance, regardless

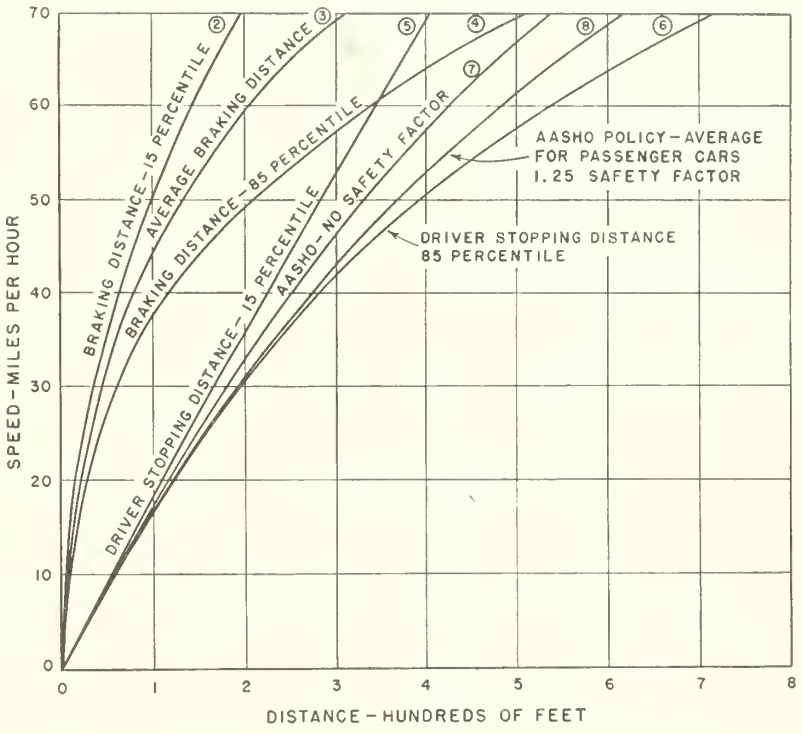
SPEED - MILES PER HOUR
DISTANCE - HUNDREDS OF FEET
BRAKING DISTANCE - 15 PERCENTILE
AVERAGE BRAKING DISTANCE
BRAKING DISTANCE - 85 PERCENTILE
DRIVER STOPPING DISTANCE - 15 PERCENTILE
AASHO - NO SAFETY FACTOR
AASHO POLICY - AVERAGE FOR PASSENGER CARS 1.25 SAFETY FACTOR
DRIVER STOPPING DISTANCE 85 PERCENTILE

Figure 10.—Relation of braking and stopping distances to speed.

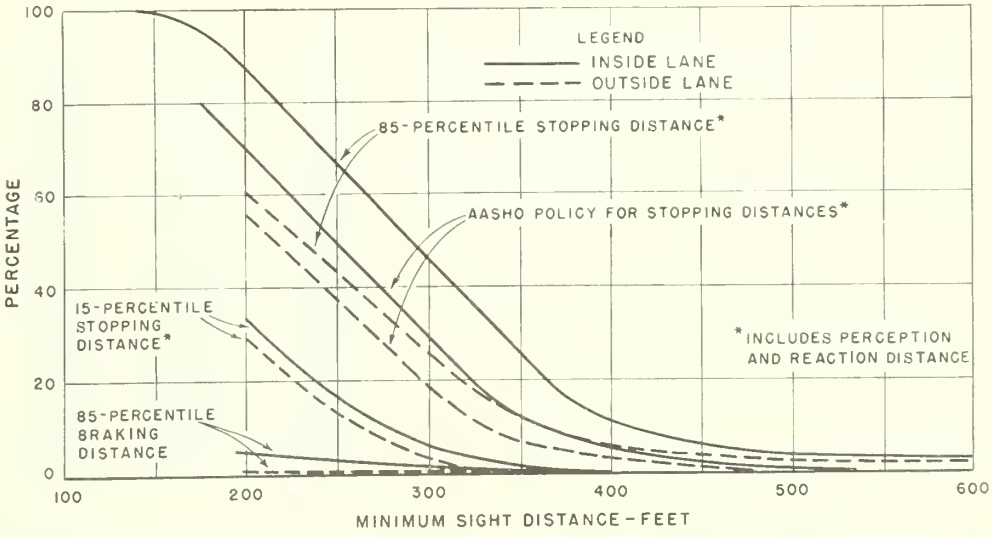

Figure 11.—Percentage of vehicles exceeding the speed from which drivers can brake or stop within available sight distance.

[3] *Praking distances of vehicles from high speeds and tests of friction coefficients*, by O. K. Normann. PUBLIC ROADS, vol. 27, No. 8, June 1953.

of which criterion is used. At locations having sight distances shorter than 400 feet, however, there is wide divergence in the percentages of vehicles traveling in excess of those speeds which conform to the stopping distances established by the various criteria being tested. Where the minimum sight distance is 200 feet, for example, 86 percent of the drivers in the inside lanes exceed the speed from which 85 percent could have stopped safely. Stated another way, 86 percent of the drivers exceed the speed from which 15 percent could not have stopped. This does not necessarily indicate an unsafe condition if it could be assumed that the faster drivers were those with the better brakes, or were more alert and consequently might have shorter-than-average perception and reaction time. It strongly suggests, however, that few drivers could have stopped had an object suddenly come into view in their lane.

With a minimum sight distance of 200 feet, 31 percent of the drivers in the inside lanes exceed the speed from which only the best 15 percent could stop. This can scarcely be considered safe operation. At this minimum sight distance, 70 percent of the drivers in the inside lanes exceed the design speed based on the A.A.S.H.O. standards for nonpassing sight distances only. For each of the criteria, the percentage of vehicles exceeding the safe speed is considerably lower for vehicles in the outside lanes than in the inside lanes.

Returning to a sight distance of 400 feet, 10 percent of the drivers exceed the speed from which 85 percent of the vehicles could have stopped in less than 400 feet. Only 5 percent of the vehicles exceed the speed from which they could stop within the minimum nonpassing stopping distance recommended by the A.A.S.H.O. It is apparent, therefore, that when the minimum sight distance is 400 feet or longer few drivers observed in these studies overdrove the sight distance on horizontal curves based on any of the criteria. When sight distances are shorter than 400 feet, the percentage of drivers exceeding the speeds which permit braking within the available sight distance, with allowance for perception and reaction time, increases rapidly with a decrease in the sight distance. Very few of the drivers, however, under any of the conditions studied, were traveling faster than the speed from which they could have stopped within the available sight distance when braking distance alone is considered, with no allowance for driver perception and reaction time.

Speed, Sight Distance, and Curvature Related

A high correlation has been established between speed and curvature, and a relation, but one with low correlation, has been established between speed and minimum sight distance. No close relation could be found between curvature and sight distance. All three variables must be considered in combination, however, to obtain their true effect on driver behavior. The combined relation between average speed, minimum sight distance, and curvature is shown in figures 12 and 13. The study locations have been combined into several sight-distance and curvature groups giving a range of conditions which may be applicable to the normal highway in the area.

Table 6.—Speeds from which vehicles could stop within available sight distance, based on various assumed criteria, and percentages of vehicles exceeding those speeds, on inside lanes of 2-lane highways

Study site No.	Minimum sight distance	Stopping speed as determined by using available sight distance and—				Percentage of observed vehicles exceeding calculated stopping speeds based on—			
		85-percentile driver stopping distance (curve 6 [1])	A.A.S.H.O. policy for stopping distance (curve 8 [1])	15-percentile driver stopping distance (curve 5 [1])	85-percentile braking distance (curve 4 [1])	85-percentile driver stopping distance	A.A.S.H.O. policy for stopping distance	15-percentile driver stopping distance	85-percentile braking distance
	Feet	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	200	30.6	31.0	36.0	49.0	0	0	0	0
2	215	32.3	32.5	39.0	51.0	81.6	80.7	33.5	0
3	215	32.3	32.7	38.5	50.5	48.3	44.8	10.3	0
4	220	33.0	33.8	39.0	51.0	18.7	13.8	1.3	0
5	236	35.0	35.8	41.7	52.1	73.0	68.4	36.9	3.3
6	256	37.0	37.6	45.5	54.0	80.4	78.1	15.2	1.2
7	285	40.4	41.5	51.5	56.2	21.1	16.6	0	0
8	297	41.7	42.6	52.0	57.0	44.5	39.2	9.8	2.8
9	300	42.0	43.0	53.0	57.0	0	0	0	0
10	300	42.0	43.0	53.0	57.0	20.0	15.6	.4	0
11	303	42.3	43.0	53.1	57.0	73.4	70.2	20.7	9.9
12	303	42.3	43.0	53.1	57.0	44.7	39.1	3.9	.9
13	308	42.8	43.8	54.0	57.9	43.1	32.3	.8	0
14	320	43.9	45.0	56.0	59.0	15.9	10.6	0	0
15	320	43.9	45.0	56.0	59.0	14.2	5.0	0	0
16	324	44.2	45.5	58.8	57.0	42.5	32.5	2.7	3.6
17	342	46.0	47.3	60.2	60.0	25.8	19.7	0	0
18	350	46.4	48.1	61.0	61.0	24.6	16.7	.6	.6
19	360	47.3	49.0	63.0	61.6	24.5	18.4	3.3	3.9
20	371	48.2	50.0	64.4	62.3	11.5	6.9	0	0
21	377	48.7	50.6	65.2	62.6	73.3	61.2	.9	6.1
22	380	49.0	50.9	66.0	62.8	11.8	7.3	0	0
23	400	50.5	52.9	70.0	64.0	10.6	6.0	0	.1
24	407	51.0	53.7	70.0+	64.5	14.8	6.0	0	.6
25	420	52.0	54.7	70.0+	65.3	2.3	.9	0	0
26	435	53.1	55.9	70.0+	65.0	.6	0	0	0
27	440	53.5	56.0	70.0+	66.0	11.1	6.8	0	0
28	450	54.2	57.0	70.0+	66.2	10.5	5.4	0	0
29	460	55.0	58.0	70.0+	67.5	12.0	6.6	1.0	0
30	469	55.4	58.9	70.0+	67.9	2.3	.9	0	0
31	470	55.6	59.0	70.0+	68.0	1.8	.4	0	0
32	490	56.9	60.5	70.0+	65.3	5.1	0	0	1.7
33	500	57.6	61.1	70.0+	69 5	3.9	3.9	0	1.0
34	500	57.6	61.1	70.0+	69.5	1.4	.6	0	0
35	510	58.2	62.0	70.0+	70.0	2.0	.7	0	0

[1] Curve numbers in fig. 10.

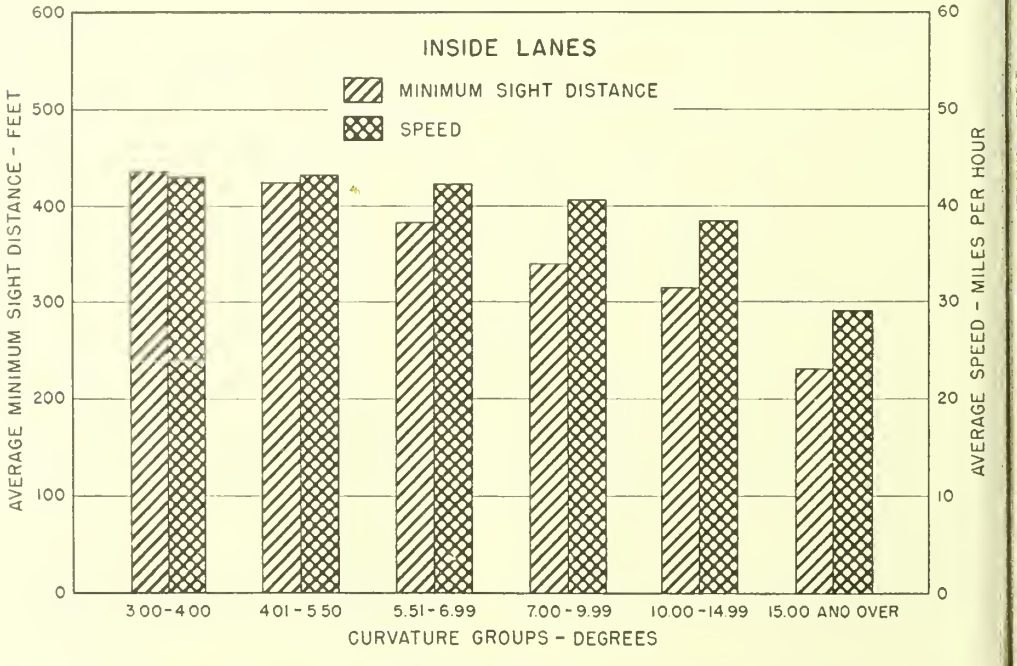

Figure 12A.—Average relation between minimum sight distance, speed, and curvature on inside lanes.

Table 7.—Speeds from which vehicles could stop within available sight distance, based on various assumed criteria, and percentages of vehicles exceeding those speeds, on outside lanes of 2-lane highways

Study site No.	Minimum sight distance	Stopping speed as determined by using available sight distance and—				Percentage of observed vehicles exceeding calculated stopping speeds based on—			
		85-percentile driver stopping distance (curve 6 [1])	A.A.S.H.O. policy for stopping distance (curve 8 [1])	15-percentile driver stopping distance (curve 5 [1])	85-percentile braking distance (curve 4 [1])	85-percentile driver stopping distance	A.A.S.H.O. policy for stopping distance	15-percentile driver stopping distance	85-percentile braking distance
	Feet	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
4	220	33.0	33.8	39.0	51.0	26.5	19.4	1.8	0
2	220	33.0	33.8	39.0	51.0	80.3	76.4	45.5	0
1	300	42.0	43.0	53.0	57.0	0	0	0	0
30	316	43.7	44.8	55.6	58.2	34.9	26.3	2.2	1.9
14	320	43.9	45.0	56.0	59.0	10.4	6.2	0	0
8	323	44.1	45.2	56.2	59.2	34.9	28.0	0	0
10	325	44.5	45.4	59.5	57.0	5.2	3.5	0	0
15	340	45.7	47.2	60.0	61.2	.9	.5	0	0
19	360	47.3	49.0	63.0	61.6	5.8	3.6	0	.1
18	375	48.5	51.5	65.4	62.5	17.3	7.6	0	0
13	377	48.7	50.6	65.2	62.6	2.4	1.9	0	0
22	380	49.0	50.9	66.0	62.8	5.8	2.2	0	0
7	391	50.0	52.0	68.5	62.5	1.7	1.0	0	0
23	400	50.5	52.9	70.0	64.0	11.5	6.5	0	0
31	430	52.8	55.2	70.0+	65.9	2.7	0	0	0
17	439	52.9	57.0	70.0+	66.1	5.8	1.1	0	0
27	440	53.5	56.0	70.0+	66.0	0	0	0	0
5	452	54.2	57.2	70.0+	66.2	0	0	0	0
16	452	54.2	57.2	70.0+	66.2	1.3	0	0	0
6	453	54.3	57.3	70.0+	66.3	1.1	.4	0	0
3	455	54.2	57.0	70.0+	67.0	0	0	0	0
26	460	55.0	58.0	70.0+	67.5	1.2	.5	0	0
29	460	55.0	58.0	70.0+	67.5	27.4	10.0	0	1.0
12	489	57.0	62.0	70.0+	70.0+	1.8	0	0	1.1
34	500	57.6	61.1	70.0+	69.5	3.0	1.2	0	.1
32	500	57.6	61.1	70.0+	69.5	1.6	0	0	0
35	520	58.9	63.0	70.0+	70.0+	1.9	0	0	0
11	526	59.2	63.4	70.0+	70.0+	2.4	0	0	0
33	530	59.4	63.7	70.0+	70.0+	3.8	.7	0	0
21	546	60.5	65.0	70.0+	70.0+	3.5	.7	0	0
24	550	60.9	65.0	70.0+	70.0+	1.8	.7	0	0
28	557	61.6	66.5	70.0+	70.0+	2.2	0	0	0
20	655	66.5	70.0+	70.0+	70.0+	0	0	0	0

[1] Curve numbers in figure 10.

Figure 12A shows, for the inside lanes on the curves, the manner in which average speed and minimum sight distance vary with the degree of curvature. It will be noted that the average minimum sight distance decreases as the curvature increases. The average speed also has a tendency to decrease with an increase in curvature, but speed is not materially affected by changes in curvature between the limits of 3 and 10 degrees. For curves of 10 degrees or more, a significant reduction in speed accompanies an increase in the degree of curvature.

Figure 12B shows comparable information for the outside lanes. Neither the minimum sight distance nor the average speed in the outside lanes is reduced appreciably as the curvature increases to 10 degrees. The average speed is reduced to about 28 miles per hour on curves of 20 degrees or over where the average minimum sight distance is 325 feet.

Figure 13 shows separately for the inside and outside lanes the average curvature and speed as related to the minimum sight distance. For the inside lanes, the average curvature decreases without exception with each increment of minimum sight distance. The average speed, however, increases from 34 miles per hour to something over 40 miles per hour with a change in minimum sight distance from 200 to 350 feet, while at the same time the curvature decreases from 16 degrees to 7 degrees. Neither average curvature nor speed show marked change as the minimum sight distance increases above 350 feet.

Conditions for the outside lanes are somewhat at variance with those for the inside lanes. While changes in sight distance are accompanied by changes in degree of curvature in much the same manner as for the inside lanes, average speeds in the outside lanes are progressively higher for successive increases in minimum sight distance throughout the sight distance range.

Figures 12 and 13 show the average speeds obtained on horizontal curves having curvatures and sight distances varying within certain fixed limits. There is considerable range in sight distance on curves having the same degree of sharpness, however, and different locations having about the same minimum sight distance have a considerable range in the sharpness of the curvature.

Figure 14 shows the average, 90-percentile, and 95-percentile speeds, respectively, for horizontal curves throughout the range of curvature and sight-distance conditions included in this study, and indicates the speeds that would most likely be found on horizontal curves on existing two-lane highways with the conditions of speed and sight distance generally prevailing in the areas included in this study. The relations shown were obtained by applying the method of least squares to the basic data for the inside and outside lanes combined. The curves in this figure represent speed contours. Shown on each part of the figure is the standard error within which the speeds can be assumed to be correct.

This figure shows that the sight distance has a comparatively small effect on vehicle speeds, whereas curvature has a considerable effect. With a constant curvature (reading horizontally) the average change in speed is about 0.8 mile per hour for each 100-foot change in sight distance. With a constant sight distance (reading vertically) the average

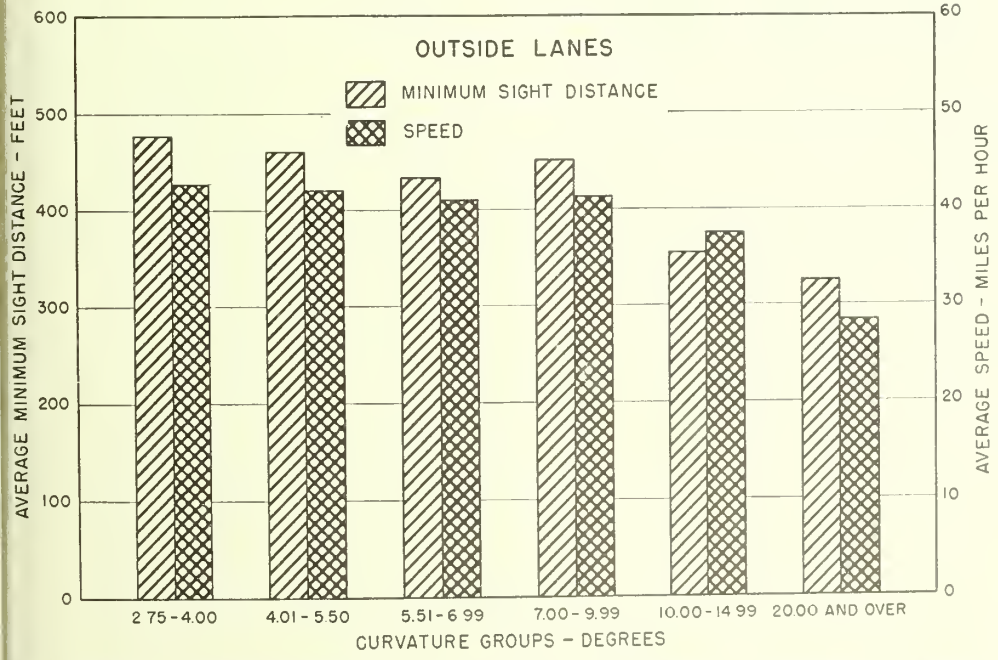

Figure 12B.—Average relation between minimum sight distance, speed, and curvature, on outside lanes.

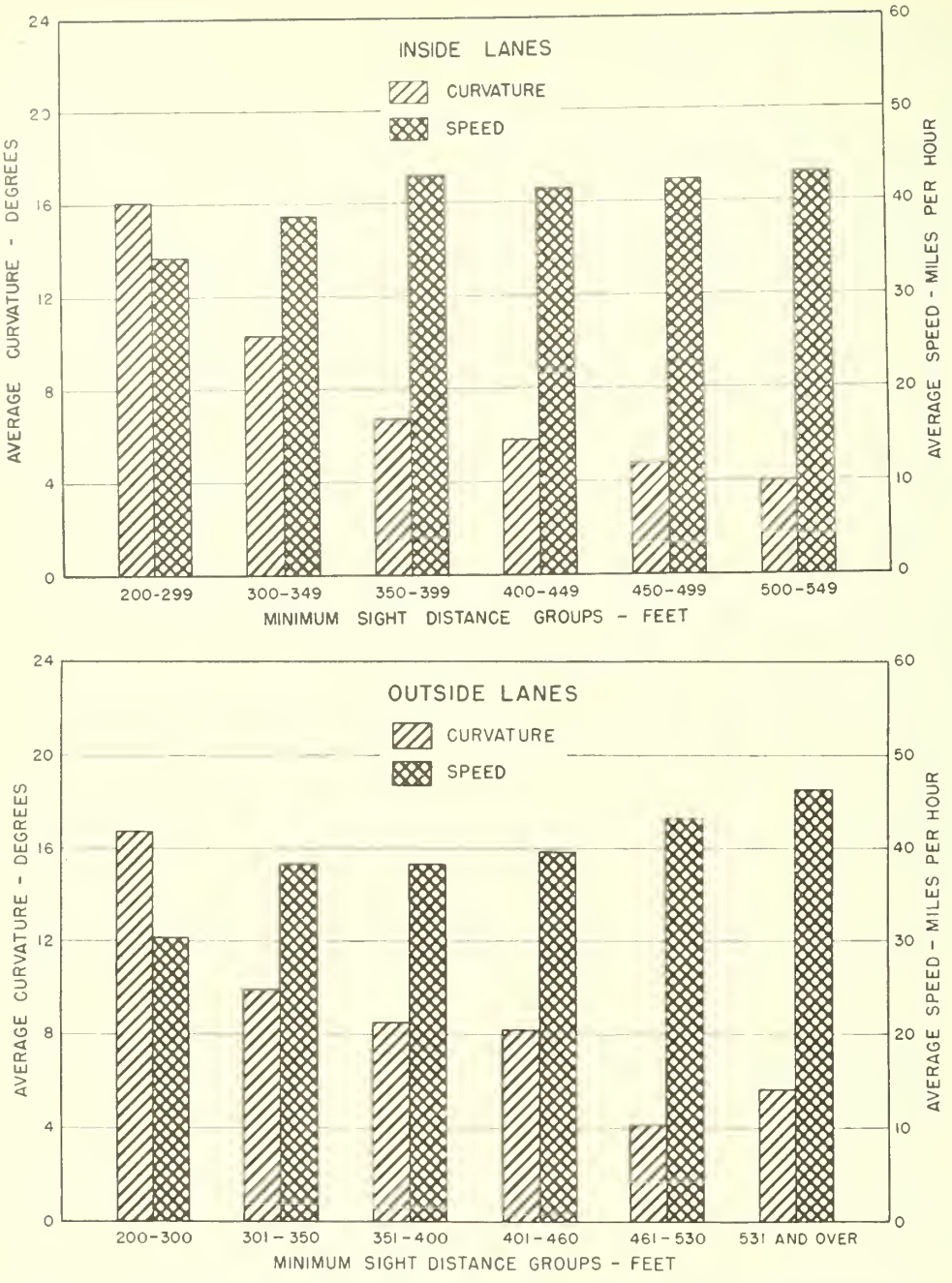

Figure 13.—Average relation between curvature, speed, and minimum sight distance.

speed changes uniformly about 0.7 mile per hour for each 1-degree change in curvature. As previously stated, however, the relation between sight distance and curvature is such that a 3-degree change in curvature is approximately equivalent to a 100-foot change in sight distance. Then, since a 3-degree change in curvature causes a 2.1-mile per hour change in average speed (with sight distance constant) and a 100-foot change in sight distance causes an 0.8-mile per hour change in average speed (with curvature constant), curvature causes nearly three times as great a change in speed as sight distance, under comparable conditions. This is as true for the 90- and 95-percentile speeds as for the average speed.

Comparison of Driver Behavior on Horizontal and Vertical Curves

Along with this study, carried out as a cooperative project by the Bureau of Public Roads and the New York State Department of Public Works, a companion study on vertical curves was included in the program.[4] It is interesting to compare the results of these two studies, as shown in table 8.

It will be noted that with the same minimum sight distance, vehicle speeds are considerably lower on horizontal curves than on vertical curves. The difference in speed is greater when the sight distance is short than when it is long. This tends to confirm the conclusion that sight distance has only a minor influence on speeds on horizontal or vertical curves. If sight distance were the controlling factor there would be the same reduction in speed with a reduction in sight distance on the vertical curves as on the horizontal curves. Since this was not the case, it is apparent that the primary influencing factor was centrifugal force which is present on horizontal but not on vertical curves.

[4] See footnote 1, p. 27.

Table 8.—Comparison of vehicle speeds on vertical and horizontal curves having the same minimum sight distances

Minimum sight distance [1]	Average speed on—		95-percentile speed on—	
	Vertical curves	Horizontal curves	Vertical curves	Horizontal curves
Feet	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*
200	42	30	54	37
300	45	37	56	46
400	46	41	57	51
500	46	43	58	54

[1] Sight distanee in both cases measured to a 4-inch object

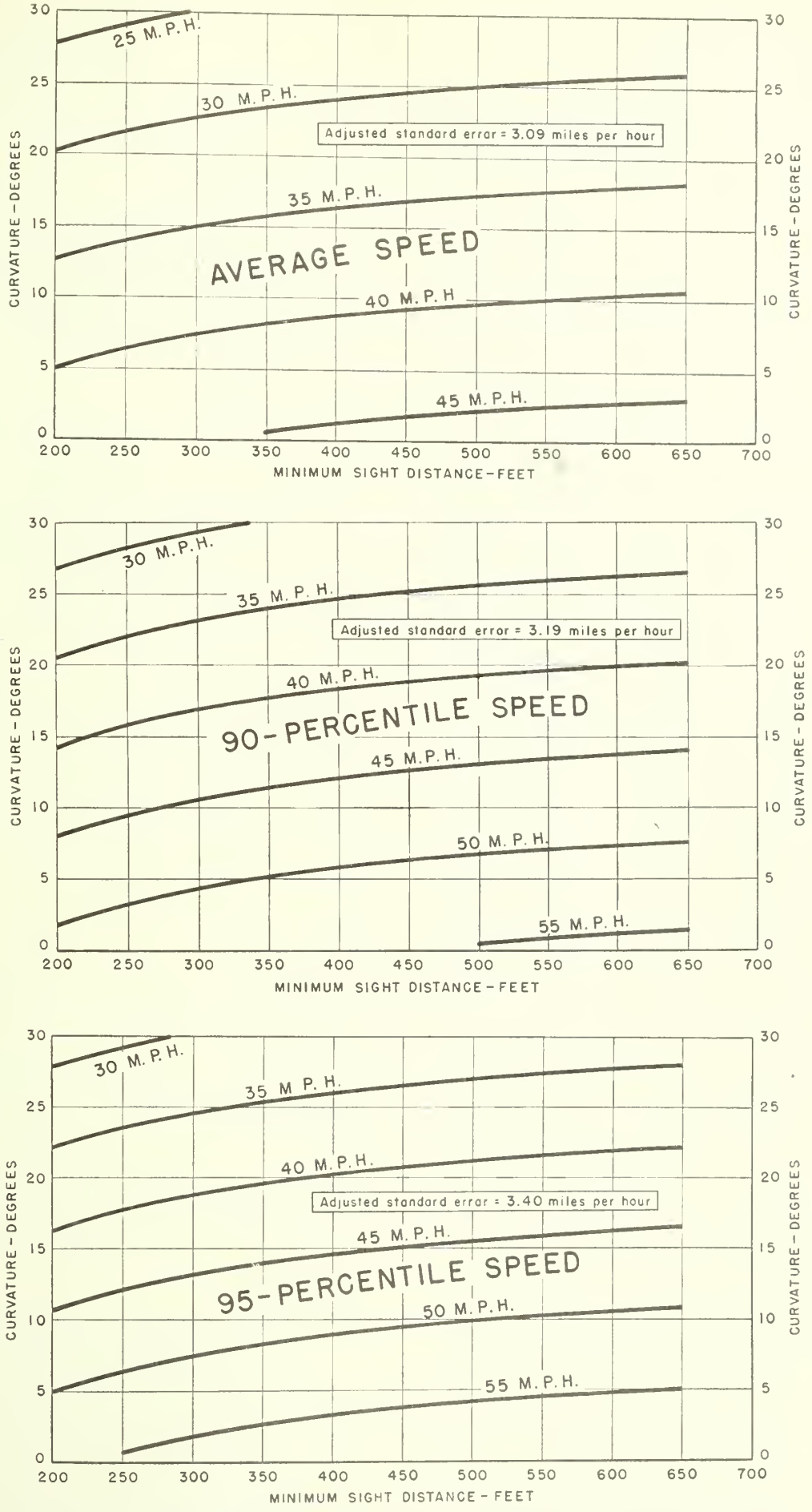

Figure 14.—Relation between minimum sight distance, curvature, and average speed.

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AS OF APRIL 30, 1954

FOR OFFICIAL USE

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$7,090	$16,258	$8,487	334.1	$8,936	$4,531	168.0	$44,621	$22,201	511.5	$69,815	$35,219	1,013.6
Arizona	2,888	6,174	4,405	127.2	1,482	1,052	37.5	6,513	4,567	83.3	14,169	10,024	248.0
Arkansas	7,858	8,413	4,709	360.4	5,299	2,658	140.9	14,542	7,252	301.7	28,254	14,619	803.0
California	8,406	24,430	11,715	163.2	20,901	10,690	91.8	83,978	41,128	199.7	129,309	63,533	454.7
Colorado	10,285	2,764	1,533	45.8	1,241	692	41.8	19,227	10,538	213.9	23,232	12,763	296.5
Connecticut	9,871	5,238	2,464	7.6	251	125	.2	9,256	4,611	26.8	14,745	7,200	34.6
Delaware	2,333	2,673	1,363	12.2	2,500	1,601	18.3	4,652	2,319	24.4	9,825	5,283	54.9
Florida	4,894	25,614	12,969	406.4	4,558	2,369	68.7	20,607	10,806	316.3	50,779	26,144	791.4
Georgia	15,078	11,985	6,127	196.7	7,425	3,516	132.4	39,392	18,717	596.6	58,802	28,360	925.7
Idaho	2,050	8,175	5,189	179.6	6,398	3,993	100.1	12,778	7,925	218.5	27,351	17,107	498.2
Illinois	15,493	36,038	19,152	352.3	23,549	11,671	186.7	63,538	34,482	347.8	123,125	65,305	886.8
Indiana	11,768	42,878	22,823	201.3	16,708	8,375	107.2	20,562	10,795	94.4	80,148	41,993	402.9
Iowa	4,193	17,103	9,084	705.5	11,694	6,228	427.1	19,294	10,608	600.4	48,091	25,920	1,733.0
Kansas	8,580	11,919	5,995	1,041.8	6,395	3,301	325.3	14,873	7,435	651.3	33,187	16,731	2,018.4
Kentucky	7,989	11,525	6,137	167.9	6,638	3,323	72.5	22,294	11,560	238.6	40,457	21,020	479.0
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Oregon	3,473	4,303	2,534	46.1	4,028	2,427	72.1	12,870	7,790	177.8	21,201	12,751	296.0
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Virginia	6,782	12,394	5,977	183.2	4,552	2,275	103.4	33,884	16,172	226.5	50,830	24,424	513.1
Washington	1,591	20,584	10,449	277.2	4,137	2,136	96.2	16,644	8,836	119.8	41,365	21,421	493.2
West Virginia	6,625	10,605	5,358	56.9	1,096	562	.8	16,823	8,370	59.5	28,524	14,290	117.2
Wisconsin	4,488	22,064	11,485	247.6	7,848	3,923	204.2	24,717	12,330	216.7	54,629	27,738	668.5
Wyoming	845	6,850	4,451	151.5	1,443	934	85.4	8,575	5,394	159.4	16,868	10,779	396.3
Hawaii	3,045	2,497	1,212	4.3	182	87	.5	11,596	5,510	17.3	14,275	6,809	22.1
District of Columbia	4,320	7,230	3,375	5.0	30	15	.5	12,613	5,814	2.2	19,873	9,204	7.7
Puerto Rico	6,539	10,441	4,809	45.2	989	482	5.2	15,373	7,160	44.0	26,803	12,451	94.4
TOTAL	369,636	767,437	402,722	13,323.0	371,288	188,229	6,727.2	1,377,340	694,647	12,475.4	2,516,065	1,285,598	32,525.6

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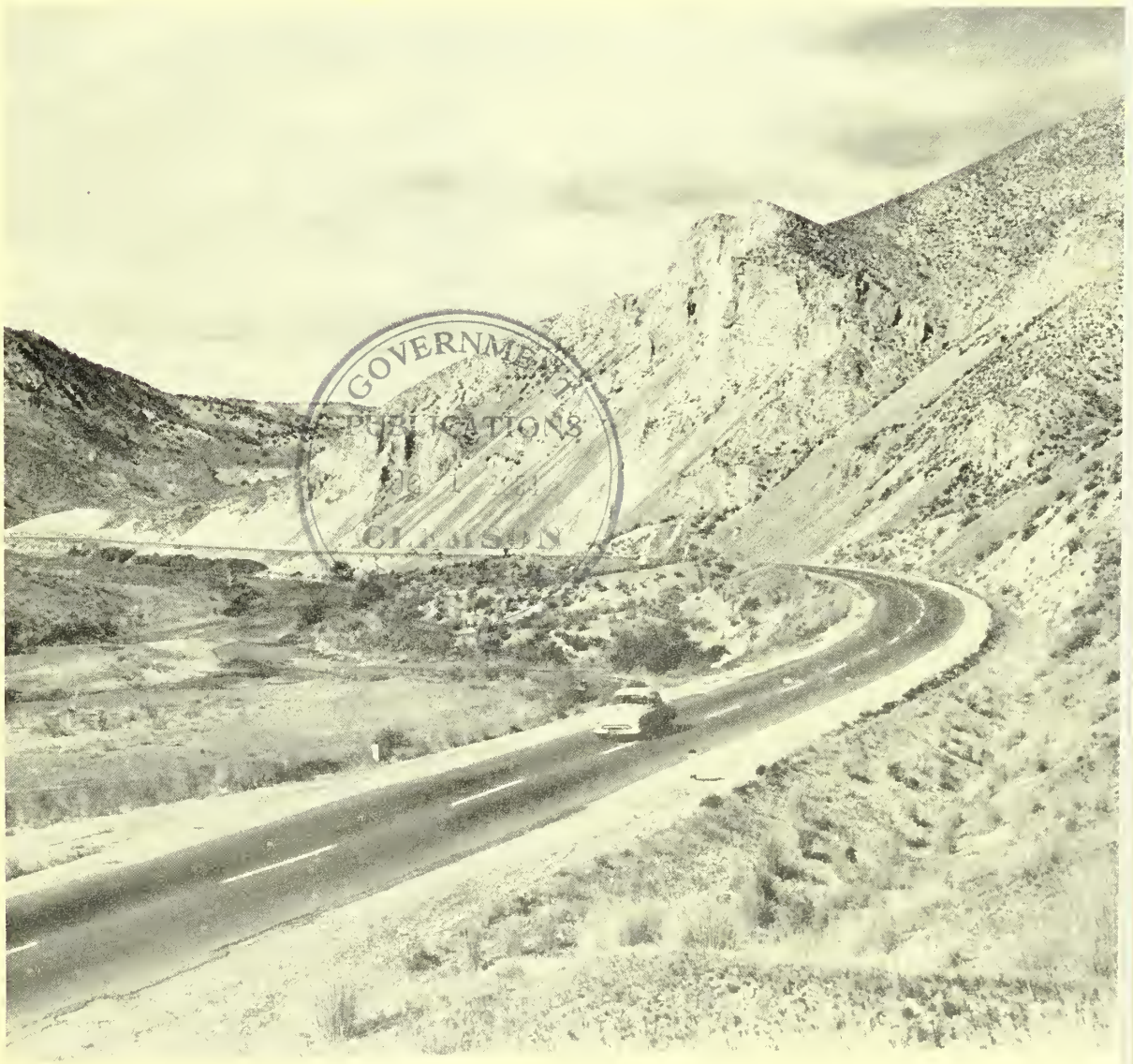

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Heat-Resistant Properties of Additives for Bituminous Materials

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by PAUL F. CRITZ, Supervising Highway Physical Research Engineer

In past years, it was generally believed that hot-mixed, hot-laid bituminous pavements were not subject to film stripping in the presence of moisture. Investigations made where failures of such bituminous pavements have occurred indicated, however, that film stripping had taken place in the underlying courses of the pavements, the condition being more severe in the case of impermeable bases than in those having porous granular materials. To counteract this deficiency, the use of commercial additives for treating asphalt was adopted. The results were not always satisfactory. It was felt by many authorities that the additives might not be stable at the temperatures used in constructing hot-mix bituminous pavements or when held at relatively high temperatures during shipment from the refinery. As a result, manufacturers undertook the production of additives having heat-resistant qualities for the treatment of asphalt.

This study was undertaken to test the heat-resistant properties of three additives. It was found that prolonged heating of treated asphalt at high temperatures prior to mixing reduced the effectiveness of the additives as measured by the immersion-compression test method, and that it was more detrimental to small masses than to large ones. Overheating the aggregate prior to mixing reduced the effectiveness even more. With few exceptions, the additives were found to be effective in increasing the percentage of retained strength and in reducing moisture absorption and volumetric swell. If proper attention is given to the control of temperatures of asphalt and especially the aggregate, the additives studied may be expected to reduce the detrimental effect of moisture absorption.

THE value of certain chemical agents as aids in coating aggregates with bituminous materials and increasing the resistance of the bituminous film to stripping has been fairly well established and accepted for cold-mix and cold-laid types of mixtures. Laboratory studies [1] and field investigations have shown that a number of such agents are of considerable benefit for the treatment of liquid asphaltic materials to be used with hydrophilic aggregate or with wet aggregate.

The possible value of these additives or the necessity for using them in hot-mixed bituminous mixtures was not considered in the earlier studies because, at that time, the general feeling existed that mixtures prepared hot with viscous bituminous materials, such as asphalt cement, were not liable to film stripping in the presence of moisture. However, investigations of instances of unsatisfactory service behavior of bituminous pavements revealed that extensive film stripping has occurred in many cases in the underlying courses of hot-mix, hot-laid bituminous pavements. This condition was found most often in courses laid on impermeable bases such as portland cement concrete or brick pavement, and less frequently in pavements laid on porous granular bases. To counteract this unsatisfactory performance, additives have also been used for treating asphalt for use in the hot-mix types of asphaltic pavements. Generally, the additives are added to the asphalt at the refinery.

So far, their use in hot-plant mixtures has not always been satisfactory. The results have been variable in many cases and often inconsistent. On one important experimental project, several additives that had shown value in laboratory studies were used. Later examination of the pavement indicated that stripping had occurred and that it was more dependent on the type of aggregate and the type of base on which the surfacing was laid than on the additive used. Informal reports indicate that similar results have been obtained in other localities. The consensus of opinion has been that additives, as currently produced, may not be stable at the temperatures used in constructing hot-mix bituminous pavements or possibly they do not remain stable when held at relatively high temperatures during shipment from the refinery and prior to use on the job.

As a result of these developments, manufacturers undertook the production of additives for the treatment of asphalt that would be resistant to the heat to which they would be subjected during shipment and plant mixing. Materials, for which such properties were claimed, were later placed on the market.

To determine to what extent these materials are heat resistant, the Bureau of Public Roads conducted a study in which asphalt cement treated with an additive, and asphalt cement not treated, were heated to two different temperatures and then combined with aggregates which also had been heated to two different temperatures. The resultant mixtures were then formed into test specimens which were subjected to the immersion-compression test.

Conclusions

The results of this investigation, with respect to the three additives which were tested with asphalt from one source, are summarized as follows:

1. Prolonged heating of the treated asphalt at high temperatures prior to mixing reduced the effectiveness of the additives. For these tests, heating at 325° F. for 72 hours practically destroyed the effectiveness of the additives as measured by the results of the immersion-compression test. However, it is known that the effect of heating is much more detrimental to small masses of asphalt than to large ones. Therefore, it may be concluded that, for the same temperatures and periods of heating that were used in these tests, much less severe reductions in the effectiveness of the additives would take place in practice where the asphalt is held at elevated temperatures in large masses.

2. Overheating the aggregate reduced the effectiveness of the additives more than did prolonged heating of the treated asphalt prior to mixing. It may be noted that mixing operations, as performed in connection with this investigation, much more nearly simulated the conditions of practice than did the preliminary heating of the asphalt.

3. Even under the severe conditions of these tests the additives, with only a few exceptions, were effective in specimens subjected to the immersion-compression test in increasing the percentage of retained strength and in reducing the moisture absorption and volumetric swell.

It is concluded that if proper attention is given to the control of the temperature of the asphalt and the aggregate, particularly the

[1] *Additives for bituminous materials*, by P. F. Critz, PUBLIC ROADS, vol. 24, No. 5, July-Aug.-Sept. 1945; and *Further developments and application of the immersion-compression test*, by J. T. Pauls and J. F. Goode, PUBLIC ROADS, vol. 25, No. 6, Dec. 1948.

Table 1.—Physical properties of asphalt cement with and without additives, before and after heating

Additive in asphalt	Heating period		Physical property			Changes in physical properties	
	Temperature	Time	Penetration at 77° F.	Ductility at 77° F.	Softening point	Penetration	Softening point
	°F.	Hours		Cm.	°F.	Points	°F.
None	(1)	------	88	250+	118.0	------	------
A	(1)	------	116	200+	112.5	------	------
B	(1)	------	108	200+	113.4	------	------
C	(1)	------	115	180	113.2	------	------
None	250	24	78	200+	121.0	−10	+3.0
A	250	24	98	200+	116.0	−18	+3.5
B	250	24	91	200+	117.6	−17	+4.2
C	250	24	100	200+	116.5	−15	+3.3
None	250	48	71	200+	124.3	−17	+6.3
A	250	48	89	200+	117.0	−27	+4.5
B	250	48	80	200+	119.8	−28	+6.4
C	250	48	86	200+	119.5	−29	+6.3
None	250	72	63	200+	125.5	−25	+7.5
A	250	72	83	200+	119.0	−33	+6.5
B	250	72	69	200+	125.0	−39	+11.6
C	250	72	77	190	123.0	−38	+9.8
None	325	24	70	200+	125.0	−18	+7.0
A	325	24	78	200+	121.0	−38	+8.5
B	325	24	74	185	121.8	−34	+8.4
C	325	24	80	200+	120.8	−35	+7.6
None	325	48	54	200+	130.3	−34	+12.3
A	325	48	61	200+	129.0	−55	+16.5
B	325	48	56	200+	129.0	−52	+15.6
C	325	48	63	200+	128.3	−52	+15.1
None	325	72	32	17	149.8	−56	+31.8
A	325	72	59	200+	130.0	−57	+17.5
B	325	72	33	17	150.0	−75	+36.6
C	325	72	33	17	150.3	−82	+37.1

[1] Original test—before subjecting to heat.

latter, the additives studied in this investigation may be expected to reduce the detrimental effects of moisture in hot-mixed asphaltic pavements containing hydrophilic aggregates.

Plan of the Experiment

Three commercial additives, designated in this report as *A*, *B*, and *C*, respectively, were used with an asphalt cement of 88 penetration in asphaltic concrete mixtures which were subjected to the immersion-compression test (ASTM test method D1075–49T). For comparative purposes, mixtures identical with these, except that they contained no additives, were also tested.

The immersion-compression test measured the loss of cohesion resulting from the action of water on compacted bituminous mixtures. An index of the loss was obtained by comparing the compressive strength of specimens that were immersed in water with that of specimens which were not immersed.

The asphalt cement, with and without additives, was subjected to two different temperatures, 250° and 325° F., and was held at those temperatures for periods of 0, 24, 48, and 72 hours prior to the mixing operation. The aggregate was crushed granite having definitely hydrophilic properties. The amount of additive used in all instances was 2 percent by weight of the bituminous material. The basic properties of the asphalt cement with and without additives as well as the changes in these properties resulting from heating are given in table 1. The composition of the mixture used throughout this study is given below. It was designed to produce compressed specimens containing an appreciable percentage of voids in order to insure a high degree of moisture absorption since the chief property claimed for additives is that they increase the resistance of the coated aggregate to the detrimental effects of moisture.

Materials:

Aggregate: 100 percent crushed granite.

Bitumen: Asphalt cement of 88 penetration.

Additives: Designated as A, B, and C.

Table 2.—Temperatures at which molded specimens were heated

Series	Temperature, ° F.	
	Asphalt	Aggregate
I	250	300
II	250	400
III	325	300
IV	325	400

Table 3.—Results of tests of specimens with and without additives

Heating period	Dry strength				Wet strength				Retained strength				Moisture absorption				Volumetric swell			
	No additive	A	B	C	No additive	A	B	C	No additive	A	B	C	No additive	A	B	C	No additive	A	B	C
SERIES I: ASPHALT TEMPERATURE 250° F., AGGREGATE TEMPERATURE 300° F.																				
Hours	P.s.i.	P.s.i.	P.s.i.	P.s.i.	P.s.i.	P.s.i.	P.s.i.	P.s.i.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
0	286	290	300	298	107	200	205	201	37.4	69.0	68.3	67.4	4.2	2.7	3.1	3.0	2.1	0.7	1.2	1.3
24	284	341	293	291	124	195	209	182	43.7	57.2	71.3	62.5	3.6	2.6	3.0	2.8	1.8	0.6	0.9	1.2
48	287	320	312	317	95	224	224	159	33.1	70.0	71.8	50.2	3.9	2.7	3.1	3.0	3.2	0.9	1.2	1.8
72	340	344	353	316	135	214	237	167	39.7	62.2	67.2	52.8	3.4	2.6	2.9	3.2	1.9	0.7	1.4	1.7
SERIES II: ASPHALT TEMPERATURE 250° F., AGGREGATE TEMPERATURE 400° F.																				
0	331	345	347	354	176	263	257	258	53.2	76.2	74.1	72.9	3.3	2.4	2.8	2.6	1.0	0.5	0.9	1.1
24	312	350	328	321	165	235	249	225	52.9	67.2	75.9	70.1	2.9	2.7	2.8	2.3	1.1	0.9	0.6	0.8
48	332	346	354	327	207	261	240	248	62.4	75.4	67.8	75.8	2.5	2.5	2.9	2.3	1.4	0.8	0.9	1.5
72	390	360	415	359	243	244	323	230	62.3	67.8	77.8	64.1	2.6	2.2	2.6	2.8	0.8	0.4	0.9	1.2
SERIES III: ASPHALT TEMPERATURE 325° F., AGGREGATE TEMPERATURE 300° F.																				
0	280	308	322	320	113	216	207	198	40.4	70.2	64.3	61.9	4.1	2.6	3.0	3.0	2.0	0.8	1.1	1.4
24	314	356	330	322	140	220	230	210	44.6	61.8	69.7	65.2	3.3	2.5	2.7	2.1	1.7	0.6	0.8	0.6
48	386	382	384	357	162	265	229	230	42.0	69.4	59.6	64.4	3.0	2.5	3.1	2.2	2.6	1.0	1.3	1.7
72	505	404	505	478	290	250	291	280	57.4	61.9	51.5	58.6	2.7	2.3	3.0	3.0	1.4	0.5	1.7	1.6
SERIES IV: ASPHALT TEMPERATURE 325° F., AGGREGATE TEMPERATURE 400° F.																				
0	308	358	358	377	179	267	262	300	58.1	74.6	73.2	79.6	3.2	2.2	2.5	2.2	1.2	0.5	0.8	0.7
24	329	385	357	366	215	266	301	268	65.3	69.1	84.4	73.2	2.6	2.4	2.3	1.9	0.8	0.6	0.3	0.4
48	425	424	402	414	280	334	305	288	65.9	78.8	75.9	69.6	1.8	2.1	2.5	2.0	1.0	0.6	1.0	1.0
72	600	440	640	548	390	311	463	385	65.0	70.7	72.4	70.2	2.3	2.1	2.5	2.2	0.6	0.3	0.8	1.2

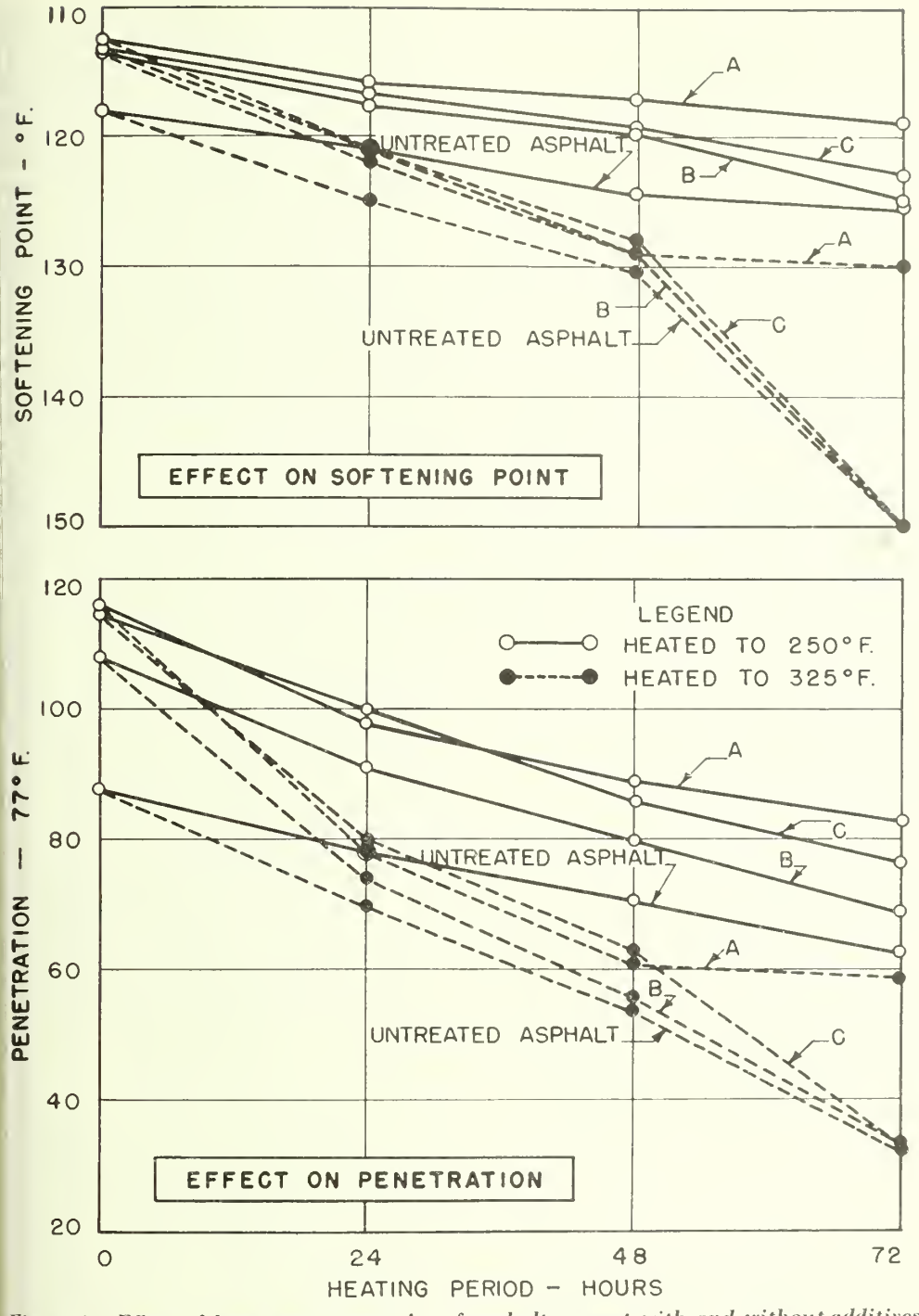

Figure 1.—Effect of heat upon properties of asphalt cement with and without additives.

Proportions:

Aggregate; gradation:

Sieve size	*Percent passing*
⅜-inch	100
No. 4	82
No. 10	36
No. 40	13
No. 80	9
No. 200	4

Bitumen: 6 percent, by weight of aggregate.

Additives: 2 percent by weight of asphalt.

Physical properties of preliminary test specimens, without additives, prepared according to standard procedure:

Density: Theoretical maximum, 2.518; actual, 2.274.

Voids: In aggregate, 21.2 percent; in compressed mixture, 9.7 percent.

Moisture absorption, 3.9 percent.

Volumetric swell, 2.2 percent.

Compressive strength, dry, 278 p.s.i.

Retained strength of immersed specimens, 43 percent.

It was obviously not possible in the laboratory to duplicate normal conditions where a large mass of material is held at a high temperature in a tank car or storage tank with a relatively small surface exposure. In this investigation the asphalt was heated in tinned containers 3 inches in diameter and 2 inches deep. Each tin contained 105 grams of asphalt, slightly more than the amount required for a single batch for two specimens. The containers were covered with loose-fitting lids to reduce oxidation. At the end of the heating period the bituminous material was immediately mixed with the aggregate which also had been brought to the desired temperature.

Two temperatures were selected to which the aggregates were heated immediately prior to mixing. They were 300° and 400° F. The higher temperature is somewhat above that recommended for good construction procedure but it is not as high as the temperature to which aggregates are often heated unless strict control is exercised. All aggregate was held overnight at 300° F. which was the heating limit of the oven. Aggregate to be used at 400° F. was raised from 300° to 400° F. on a hot plate with continuous stirring to obtain uniformity and eliminate local overheating.

Mixing was completed mechanically as rapidly as possible and from each batch, two 3-inch by 3-inch test cylinders were molded in accordance with the standard procedure used in the immersion-compression test. Three batches or six test specimens constituted one test unit. Three specimens were tested dry and three were tested after immersion in water at 120° F. for four days. The two groups of three specimens each were so selected that the average density of the specimens selected for immersion was, as nearly as possible, the same as that of the specimens to be tested dry.

The results reported are the compressive strengths of the specimens tested dry, the compressive strengths of the specimens tested wet after immersion, and the strength retained by the wet specimens expressed as a percentage of the strength of the comparable dry specimens.

For reference purposes, the specimens subjected to the four different heating temperatures are grouped into four series as shown in table 2.

No attempt was made to determine the composition of the additives. A variety of materials are available which can be used in their preparation and substitutions can be made whenever the producers find it advisable for economic or other reasons. Therefore, the actual composition of the additives used in this study is of little concern and any conclusions based upon their composition would be of no value if additives produced more recently had a different composition. Data presented and conclusions drawn herein consequently are based upon the tests of the additives made available for the study.

Effect of Additives on Asphalt

The asphalt cement used in this study had an original penetration of 88, a ductility of 250 (plus) and a softening point of 118° F. As shown by the data in table 1 and the curves in figure 1, these properties were changed when additives were combined with asphalt. For example, with 2 percent of additive A, the

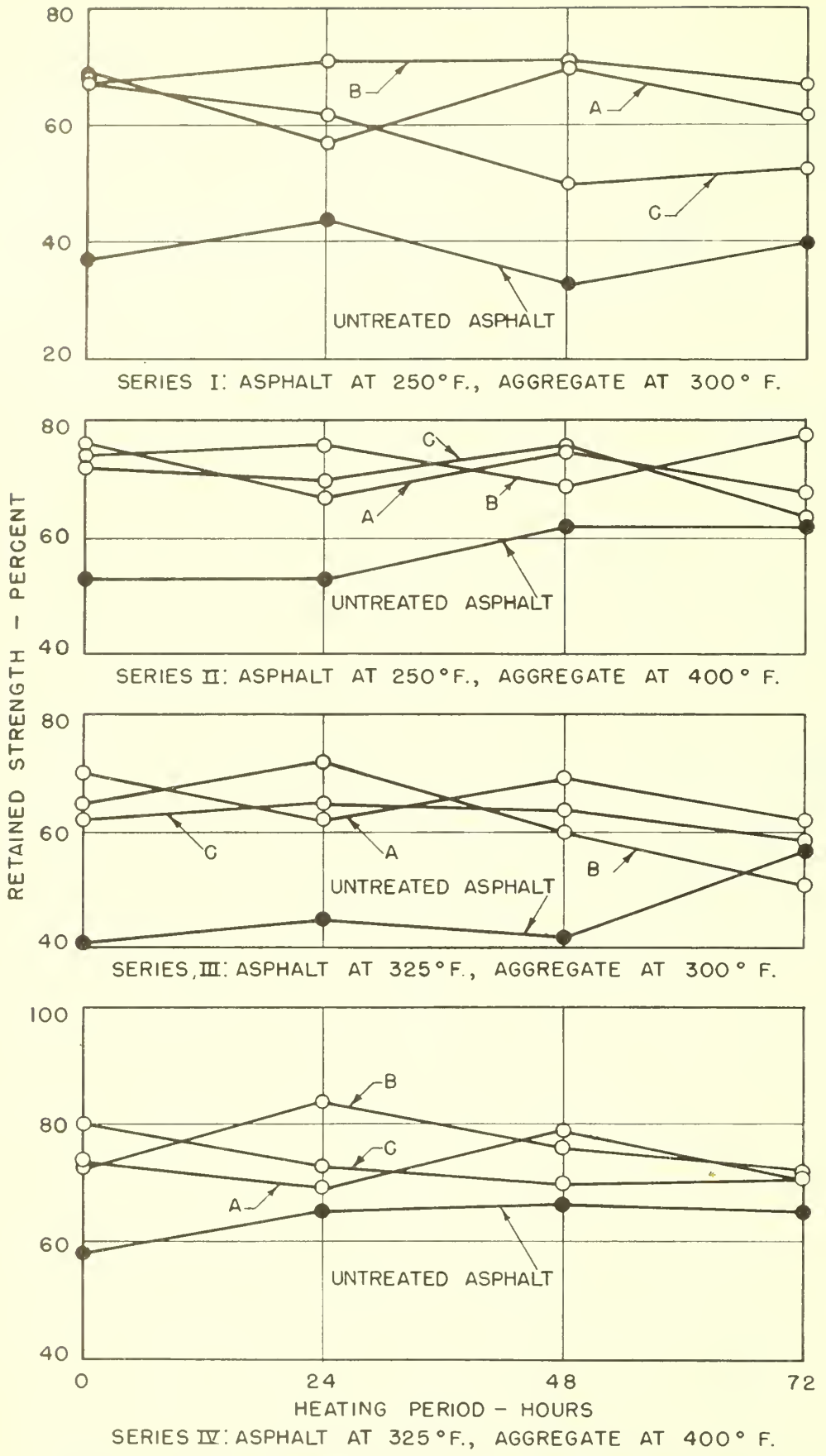

Figure 2.—Compressive strength retained after four days immersion at 120° F.

penetration was increased to 116; with additive *B*, it was increased to 108; and with additive *C*, it was increased to 115.

As the period of heating was extended and also as the heating temperature was raised, the penetration of the asphalt dropped in every instance as would normally be expected. However, for every condition of heating, the penetration of asphalt containing an additive remained higher than that of the asphalt without an additive. It will be noted from the data in table 1 and from the curves shown in figure 1 that for additive *A* the penetration remained above 50 for all conditions of test, and that for the untreated samples and those containing additives *B* and *C* the penetration remained above 50 for all conditions of test except heating at 325° F. for 72 hours when a more severe drop in penetration occurred. It will be further observed that the ductility of all samples remained high except as a result of heating to 325° F. for 72 hours. Here the material containing additive *A* retained high ductility while the other samples retained a ductility of only 17 centimeters. Similarly, the softening point of the combinations changed as heating progressed and, after 72 hours at 325° F., the samples containing no additive, and those containing additives *B* and *C*, had a softening point of 150° F., whereas the material containing additive *A* had a softening point of only 130° F. The greater resistance to hardening of asphalt treated with additive *A*, as compared with additives *B* and *C*, suggests that this additive is more resistant to deterioration caused by prolonged heating than are the other two. However, in the immersion-compression tests which are to be described, additive *A* showed no marked superiority.

In considering the results of the tests which have been discussed, it should be remembered that the treated and untreated materials were heated in very small quantities as compared with the quantities of similar materials which

Table 4.—Comparison of retained strengths of mixtures with and without additives

Heating period	Retained strength [1]			
	A	B	C	Average
SERIES I: ASPHALT 250° F. AGGREGATE 300° F.				
Hours	*Percent*	*Percent*	*Percent*	*Percent*
0	184	183	180	182
24	131	163	143	146
48	212	217	152	194
72	157	169	133	153
Avg.	171	183	152	168
SERIES II: ASPHALT 250° F. AGGREGATE 400° F.				
0	143	139	137	140
24	127	143	132	134
48	121	109	122	117
72	109	125	103	112
Avg.	125	129	124	126
SERIES III: ASPHALT 325° F. AGGREGATE 300° F.				
0	174	159	153	162
24	139	156	146	147
48	165	142	153	153
72	108	90	102	100
Avg.	147	137	139	141
SERIES IV: ASPHALT 325° F. AGGREGATE 400° F.				
0	128	126	137	130
24	106	129	112	116
48	120	115	106	114
72	109	111	108	109
Avg.	116	120	116	117

[1] Expressed as percentage of retained strength of mixture without additives.

n practice, would be heated in tank cars or storage tanks. The results show very clearly that prolonged heating produces hardening of both untreated and treated asphalts, but it is not to be expected that the hardening which would occur in tank cars or storage tanks would approach in severity the hardening which took place in the small test samples.

Immersion-Compression Tests

The results of the immersion-compression tests are given in table 3 and are shown graphically in figure 2. In table 4 the retained strengths of mixtures containing additives are compared with the retained strengths of mixtures without additives.

Reference to table 3 shows that, as the severity of the test increased, there was a general increase in the compressive strength of the specimens, both those which were tested dry and those which were tested wet after having been immersed in water for four days at a temperature of 120° F. This is attributed to the hardening of the asphalt which took place during the preliminary heating, as shown in table 1 and figure 1, and to the additional hardening which took place during the mixing operation when the asphalt in thin films was brought in contact with the hot aggregates. As has been stated, the hardening of the asphalt which took place in the laboratory during the preliminary heating was undoubtedly much greater than the hardening which would take place in practice as a result of the preliminary heating. However, the additional hardening which took place during mixing is more nearly comparable with that which would occur in practice.

A study of tables 3 and 4 and figure 2 shows that:

1. For all test conditions except one, the specimens containing asphalt treated with any one of the three additives had a higher percentage of retained strength than did those containing untreated asphalt. The one exception was in series III for asphalt treated with additive *B*, heated for 72 hours at 325° F., and used with aggregate having a temperature of 300° F.

2. With respect to percentage of retained strength, there was no consistent difference in the performance of the three additives.

3. In general, the effectiveness of the additives, as measured by percentage of retained strength, was reduced:

(a) As the period of preliminary heating was increased.

(b) As the temperature of the preliminary heating was increased. Comparison of the average values in table 4 for series I and III in which the aggregate was heated to 300° F. shows that this effect is quite marked. It is much less so in series II and IV where the aggregate temperature was 400° F., thus indicating that overheating the aggregate had a more detrimental effect than overheating the asphalt.

(c) As the temperature of the aggregate was increased. This is shown by comparison of the average values in table 4 for series I and II and for series III and IV.

The advantage afforded by the additives, as measured by the results of the immersion-compression test, was greatest when the temperature and time of the preliminary heating and the temperature of the aggregate were the least; that is, when the asphalt was brought to a temperature of 250° F. and immediately mixed with aggregate at 300° F. As the severity of the test conditions increased this initial advantage decreased until at the end of the 72-hour heating period at 325° F., it was practically eliminated.

As indicated in the previous discussion, it is fair to conclude that the loss in effectiveness of the additives, as a result of prolonged preliminary heating, will be much less severe in practice than it was in these laboratory tests. It is also concluded that overheating the aggregate is more detrimental than prolonged heating of the asphalt.

Table 5.—Comparison of the effectiveness of additives in reducing moisture absorption and volumetric swell

	Additives			
	None	A	B	C
RANGE AND AVERAGE VALUES				
Moisture absorption:	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Minimum	1.8	2.1	2.3	1.9
Maximum	4.2	2.7	3.1	3.2
Average	3.1	2.4	2.8	2.5
Volumetric swell:				
Minimum	.6	.3	.3	.4
Maximum	3.2	1.0	1.7	1.8
Average	1.5	.6	1.0	1.2
FREQUENCY OF OCCURRENCE IN 16 TESTS				
Moisture absorption (percent):				
2.0 or less	1	0	0	2
2.1–2.5	2	10	4	6
2.6–3.0	5	6	9	7
3.1–3.5	4	0	3	1
More than 3.5	4	0	0	0
Volumetric swell (percent):				
0.5 or less	0	5	1	1
0.6–1.0	5	11	9	4
1.1–1.5	4	0	5	7
1.6–2.0	4	0	1	4
More than 2.0	3	0	0	0

Effect of Additives on Bituminous Mixtures

All specimens used in this study had the same composition and were prepared and tested under a single procedure. An opportunity is afforded, therefore, to consider the effects of additives upon certain properties of the mixture, such as moisture absorption and volumetric swell.

Moisture absorption

Reference to the data given in table 3 shows that with few exceptions moisture absorption was less when additives were used than when the asphalt was untreated. The data show that only general relations can be observed with respect to the effect of additives on absorption. A further comparison of the relative effectiveness of the additives is shown in table 5. Of the three additives tested, all were of some benefit in reducing moisture absorption and additive *A* was the most effective in this respect.

Volumetric swell

Experience has indicated that volumetric swell of bituminous mixtures subjected to moisture may be expected to result in loss of stability or strength. Consequently, it would be expected that additives would reduce the tendency of mixtures to swell if they were to be effective in maintaining the structural stability of the mixtures. The changes in volume which occurred in the four-day immersion period are given in table 3, and are expressed as the percentage increase in volume on the basis of the volume of the specimens before immersion. It will be observed that, in general, the maximum swell recorded occurred in specimens that did not contain an additive and that for the same test conditions, the amount of swell, in 38 instances out of 48 was less when an additive was used. Although no definite pattern of behavior can be established, some indication of the beneficial effect of the additives can be obtained by study of the data shown. Probably a better comparison of the effectiveness of the additives in reducing volumetric swell can be obtained from table 5. The data show that all three additives were of some benefit in reducing volumetric swell and that additive *A* was the most effective.

Properties of Concrete Containing a Blend of Portland Cement and Air-Entraining Ground Blast-Furnace Slag

BY THE PHYSICAL RESEARCH BRANCH
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The results of the tests reported in this article indicate that finely ground blast-furnace slag containing an air-entraining admixture can be used advantageously in concrete as a blend with non-air-entraining portland cement. The durability of concrete containing this blend was equal or superior to that of corresponding concretes made with either air-entraining or non-air-entraining portland cements used alone or blended with natural cement. Similarly, favorable strength performance was observed except in comparison to concrete made with unblended non-air-entraining portland cement.

FINELY ground water-granulated blast-furnace slag containing a commercial air-entraining agent has been developed recently for use as a blend with portland cement in the same ratio as natural cement is commonly blended with portland cement (one bag of natural cement plus six bags of portland cement in a seven-bag batch). This report presents the results of a series of tests to determine the effect of using a blend of a type II non-air-entraining portland cement and the finely ground blast-furnace slag containing an air-entraining admixture on the strength and durability of paving concrete. For comparison, similar tests also were made on concrete containing the unblended type II portland cement, the unblended air-entraining counterpart of that cement (type IIA), and blends of each of these portland cements and natural cement. Proportions were similar to those which have been generally employed for many years in cases where blends of portland and natural cement have been used in pavement construction.

Many of the Northeastern States have been using, for the past 20 years or so, a blend of portland and natural cement in the construction of concrete pavements. The practice has been established largely on the basis of field experience which has indicated that the replacement of one bag of portland cement by one bag of natural cement in the average pavement mix will improve the resistance of the concrete to weathering without significant loss in strength. Natural cements as used in the Northeastern area are of two types: a natural cement to which there has been added a very small amount of beef tallow (0.05 to 0.10 percent), which is designated as type N by State specifications, and a natural cement which has been plasticised by adding a relatively large quantity (0.20 to 0.30 percent) of beef tallow. The former type only was used in this investigation.

Conclusions

The results of this series of tests indicate that ground blast-furnace slag containing an air-entraining admixture can be used advantageously in concrete as a blend with non-air-entraining portland cement in the proportions of approximately 14 percent ground slag to 86 percent portland cement, by volume.

For the materials used in these tests, the durability of concrete containing this blend (as measured by laboratory freezing and thawing tests) was far superior to that of the corresponding concrete containing either the type II non-air-entraining portland cement or the blend of that cement with the type N natural cement and was as good as that of the concrete containing the type IIA air-entraining portland cement and the blend of that cement with natural cement. The strength of concrete containing the blend of the type II cement and ground slag, while not as high as that of the concrete containing the type II portland cement, was, on the average, somewhat higher than the concrete containing the blend of type II portland and natural cement. Furthermore, the strength of this combination was, in all but one case, higher than that of the concrete containing the type IIA portland cement, either unblended or blended with natural cement.

Materials Used

The following materials were used in this investigation:

Portland cement, ASTM type II.

Air-entraining portland cement, ASTM type IIA, from the same plant.

Natural cement, New York State type N.[1]

Ground water-granulated blast-furnace slag containing 0.090 percent of an air-entraining admixture.

Siliceous sand from White Marsh, Md., fineness modulus, 2.80; specific gravity, 2.64 (bulk dry basis).

Crushed limestone from Riverton, Va., specific gravity, 2.78 (bulk dry basis). 1½-inch maximum size for strength specimens and 1-inch for durability specimens.

The results of chemical and physical tests on the cements are shown in table 1. The portland cements conformed to the specification requirements of the applicable ASTM specifications in all respects. The natural cement met the requirements of ASTM specification C 10–37 for soundness, time of setting, and tensile strength at 28 days. It failed to meet the minimum tensile strength of 75 p.s.i. at 7 days required by this specification. The ground blast-furnace slag met the chemical requirements of ASTM specification C 205–48 T (portland blast-furnace slag cement) for the slag constituent of this cement. It also met the 7- and 28-day tensile strength requirements of this specification but failed to attain final set at 10 hours, as required by the specification.

Combinations and Proportions

The various cement combinations and proportions used in these tests, as well as the results of tests on the plastic concrete, are shown in table 2. Mixes 1–5 were used in the strength-test specimens; mixes 6–10 were used in the specimens tested for durability.

For the mixes containing unblended portland cement (Nos. 1, 4, 6, and 9) the proportions were adjusted to give a cement content of 6.4 bags per cubic yard, which is approximately the cement factor used in concrete paving in most of the Northeastern States. In the case of the mixes with blended cement (Nos. 2, 3, 5, 7, 8, and 10) replacements for a portion of the portland cement by natural cement or ground slag were made on the basis of a blend containing 86 percent portland cement and 14 percent natural cement or ground slag, by volume. This is approximately the ratio used in practice where it is customary to substitute one 80-pound (1 cubic

[1] See Specifications, State of New York Department of Public Works, Division of Construction, dated Jan. 2, 1947, p. 73.

Table 1.—Chemical composition and physical properties of cements and ground slag

	Portland cement		Natural cement	Gr'nd slag
	Non-air-entraining	Air-entraining		
Chemical composition (in percent):				
Silicon dioxide	21.2	21.2	23.6	33.4
Aluminum oxide	5.7	5.7	5.4	10.7
Ferrous oxide	----	----	----	1.2
Ferric oxide	4.5	4.5	4.4	----
Calcium oxide	63.0	63.0	33.1	46.4
Magnesium oxide	2.1	2.2	20.9	5.3
Sulfur trioxide	1.5	1.5	1.5	.57
Manganic oxide	----	----	----	.44
Sulfide sulfur	----	----	----	1.27
Total sulfur	----	----	----	1.50
Sodium oxide	.16	.16	.28	----
Potassium oxide	.62	.64	1.52	----
Loss on ignition	1.1	1.0	9.2	[1] 1.70
Insoluble residue	.12	.16	14.5	.21
Chloroform-soluble material	.005	.008	.032	.0004
Tricalcium silicate	46	46	----	----
Dicalcium silicate	26	26	----	----
Tricalcium aluminate	8	8	----	----
Tetracalcium aluminoferrite	14	14	----	----
Calcium sulfate	2.5	2.5	----	----
Physical properties:				
Apparent specific gravity	3.17	3.19	3.05	2.94
Specific surface cm.²/gm.	1,740	1,640	----	----
Autoclave expansion—percent	.02	.04	----	----
Normal consistency—percent	23.0	23.0	31.2	29.2
Time of set (Gillmore):				
Initial—hours	5.2	5.0	2.3	[2] 3.5
Final—do	7.2	7.5	4.1	[2] 10+
Tensile strength, 1 : 3 mortar:				
At 7 days p.s.i.	375	370	[3] 40	380
At 28 days—do	490	455	[3] 190	450
Soundness (pat test)	----	----	OK	OK
Air content (mortar test) percent	10.7	17.0	[4] 13.2	[5] 15.7

[1] Gain in weight. [2] Vicat test. [3] 1 : 2 mortar.
[4] 14 percent natural cement; 86 percent portland cement by weight).
[5] 12.5 percent ground slag; 87.5 percent portland cement by weight).

bot) bag of natural cement or ground slag for ne 94-pound (1 cubic foot) bag of portland ement in a seven-bag batch. The relative uantities used were such as to maintain the ame total volume of cementitious material er unit volume of concrete as was used in the nblended mixes.

Table 3.—Results of strength tests

Mix No.	Cement combinations: Portland cement	Cement combinations: Blending material	Air content (gravimetric)	Strength tests of concrete: Flexure: Modulus of rupture [1] at 14 days	Flexure: Modulus of rupture [1] at 90 days	Flexure: Modulus of rupture [1] at 365 days	Compression (beams [2]) at— 15 days	Compression (beams [2]) at— 91 days	Compression (beams [2]) at— 366 days	Compression (cylinders [3]) at 14 days
			Pct.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
1	Type II	None	2.5	840	900	940	6,140	6,560	8,750	5,140
2	Type II	Natural cement	2.8	805	935	845	5,400	6,450	7,200	4,600
3	Type II	Ground slag	5.4	830	915	865	5,550	6,280	7,800	4,490
4	Type IIA	None	5.6	770	840	785	5,120	6,160	6,720	4,640
5	Type IIA	Natural cement	4.4	725	890	770	4,590	5,500	7,060	4,280

[1] Tests made on 6- by 6- by 21-inch beams, tested in accordance with ASTM standard method C 78–49, with sides as molded in tension. Loaded at third points of an 18-inch span. Each value average of two or three tests.
[2] Tests made on portions of beams remaining after flexure test, in accordance with ASTM standard method C 116-49. Each value average of four or six tests.
[3] Tests made on 6- by 12-inch cylinders in accordance with ASTM standard method C 39-49. Each value average of three tests.

For the base or reference mixes (Nos. 1 and 6) a ratio of fine to coarse aggregate was established to approximate an average concrete pavement mix, using crushed stone as coarse aggregate. For the materials used in these tests, the base mixes entrained about 2.5 percent air. It was found that the blending of the natural cement with the non-air-entraining portland cement (mixes No. 2 and 7) resulted in the entrainment of very little additional air. Therefore, no adjustment in sand content was made in the case of these mixes. In all the other combinations, however, a substantial increase in air content was noted so that, in each of these cases, it was necessary to reduce the fine aggregate content sufficiently to maintain the same total quantity of cementitious material per cubic yard of concrete.

All concretes had a slump ranging from 2.4 to 3.0 inches. For this consistency the air-entrained concrete required approximately 2 gallons of water less per cubic yard of concrete than was required for the non-air-entrained concrete.

Test Procedures

Because of the extensive use of blends of natural and portland cements in paving concrete in several of the Northeastern States, as well as in certain other sections of the country, it was decided to make the primary strength study an investigation of the flexural strength of the concrete and to determine the compressive strength by means of tests on modified cubes from the broken halves of the beams remaining after the flexural tests. However, as a check on the modified cube tests, 6- by 12-inch cylinders were made for test at 14 days only. The flexural test specimens were 6- by 6- by 21-inch beams, and were tested in accordance with ASTM standard method C 78–49. The modified cubes were 6 by 6 inches in cross section and were tested in accordance with ASTM method C 116–49. Flexure specimens were tested after 14, 90, and 365 days in moist storage. The broken beams were tested as compression specimens on the following days. The results of the strength tests are shown in table 3.

Specimens for durability testing (as determined by freezing and thawing in water) were 3- by 4- by 16-inch beams. Cement contents were the same as for the strength tests. However, the maximum size of the coarse aggregate was 1 inch instead of 1½ inches, with a corresponding change in the ratio of fine to coarse aggregate necessitated by the smaller maximum size (see table 2, mix Nos. 6–10).

Two groups of specimens were made. In the first group, 3- by 4- by 16-inch beams were sawed from 4- by 16- by 24-inch slabs in such a way as to provide seven beams, each having two parallel sawed surfaces. Prior to sawing, the slabs were cured under wet burlap for 7 days, followed by 28 days storage in laboratory air. After sawing, four of the test beams from each slab were placed in moist air storage to be tested as control specimens. The other three were immersed in water for 48 hours after which the freezing and thawing test was started. In the second group, individual 3- by 4- by 16-inch beams were molded in three-gang molds from the same batch of concrete used for molding the slabs. These beams were moist-cured continuously until the start of the freezing and thawing test. No individual beams for control tests were made. All specimens were frozen and thawed in water at a temperature range from 70° to −20° F. A rather slow cycle was used, approximately 7 hours being required to reduce the temperature of the concrete from 70° to −20° F. One complete cycle required 24 hours, the specimens being placed in the freezing chamber at about 4 p. m., remaining there until about 9 a. m. the following morning,

Table 2.—Cement combinations, proportions, and tests of plastic concrete

Mix No.	Combinations used, and quantities per cubic yard of concrete: Portland cement Type II	Portland cement Type IIA	Natural cement	Ground slag	Aggregate: Fine	Aggregate: Coarse [1]	Water	Tests on plastic concrete: Slump	Weight	Air content (gravimetric)
									Lb./cu. ft.	
For strength tests:	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*In.*		*Pct.*
1 [2]	602	0	0	0	1,203	2,035	271	3.0	152.2	2.5
2	517	0	72	0	1,203	2,035	267	2.7	151.6	2.8
3	517	0	0	72	1,107	2,035	253	2.8	148.2	5.4
4	0	602	0	0	1,107	2,035	254	2.8	148.3	5.6
5	0	517	72	0	1,107	2,035	260	2.8	147.4	4.4
For durability tests:										
6 [2]	602	0	0	0	1,210	1,940	287	2.4	150.3	2.3
7	517	0	72	0	1,210	1,940	285	2.6	149.7	2.7
8	517	0	0	72	1,115	1,940	268	2.8	145.9	5.7
9	0	602	0	0	1,115	1,940	267	3.0	146.2	6.1
10	0	517	72	0	1,115	1,940	273	2.6	146.9	4.9

[1] Maximum aggregate size was 1½ inches for strength tests and 1 inch for durability tests.
[2] Base or reference mixes.

and then being placed in the thawing tank until 4 p. m.

The natural frequency of vibration of each specimen, expressed as N^2 (natural frequency of vibration, squared), was determined sonically at 70° F. at the start of the test and at periodic intervals thereafter. Alternate freezing and thawing was continued for 200 cycles or until the specimens showed a drop in N^2 of 40 percent, whichever occurred first. The reduction in N^2 at periodic intervals, expressed as percentages of the corresponding values at zero cycles, is shown in table 4. Also shown are the durability factors and the reductions in flexural strength expressed as percentages of the strength of moist-cured specimens tested at the same time. The durability factors were calculated from the following formula:

$$DF=\frac{(100-R)N}{200}$$

Where

DF=durability factor.

R=reduction in N^2 (value will be 40 percent or less).

N=number of cycles at which R was reduced 40 percent, or 200, whichever occurred first.

This formula is the same as that shown in ASTM tentative specification C 260–50 T, except that values for N^2 were used in place of the dynamic modulus of elasticity and calculations were based on a reduction in N^2 of 40 percent instead of 30 percent.

Discussion of Results

The results of the tests are summarized in table 5 which shows, for each test age, the relative strength and durability of each combination as compared to the base mix (concrete containing the unblended type II portland cement). The trends revealed by these data will be discussed briefly, first with respect to strength and second with respect to durability.

Reference to table 3 shows that, except in two cases, the base mix (No. 1) developed higher strengths both in flexure and compression than any of the other combinations. Reductions in strength (table 5) varied from an average of about 7 percent for the blend of type II portland cement and ground slag (No. 3) to an average of about 16 percent in the case of the blend of type IIA portland cement and natural cement (No. 5). The blend of type II portland cement and natural cement (No. 2) and the unblended type IIA portland cement (No. 4) showed average percentage reductions intermediate between these extremes. Reductions in strength shown for mix Nos. 4 and 5 are in line with what would be expected due to the relatively high air content of these combinations. However, mix No. 3, with next to the highest air content, showed the smallest average reduction in strength—actually less than mix No. 2, which had an air content only 0.3 percent higher than the base mix.

It will be noted from table 3 that all of the combinations except the base mix showed retrogression in flexural strength at 1 year as compared to the strength at 90 days. This is somewhat surprising, especially in view of the fact that a type II cement was used as the portland cement constituent in all of the blends as well as in the unblended air-entrained portland cement concrete. This tendency also has been noted in another current investigation of the Bureau where the effect of air-entraining admixtures on strength is being investigated. However, it was not evident in the study of age-strength relations for air-entrained concrete recently reported in PUBLIC ROADS,[2] nor was it evident in the study of various natural cement blends published in PUBLIC ROADS in 1947.[3]

In contrast with the flexural strength results, the compressive strength of all combinations increased between 91 and 366 days although the rate of increase was somewhat less for the blends and the unblended air-entrained concrete than for the base mix. From table 3, it will also be noted that the results of the 14-day compression tests of 6- by 12-inch cylinders, while considerably lower, parallel in general the 15-day tests on the modified cubes.

The three combinations which contained substantial amounts of entrained air (Nos. 8, 9, and 10) were much more resistant to alternate freezing and thawing than the two mixes (Nos. 6 and 7) with relatively low air content, as may be seen in tables 4 and 5. The average of the durability factors for the three mixes of high air content was about 65 as compared to an average value of about 30 for the two mixes of low air content. Except for mix No. 7, test specimens sawed from the 4- by 16- by 24-inch slabs showed higher resistance than the corresponding molded beams. This was due, no doubt, to the 28-day drying period introduced during the curing of the slabs. The molded beams were continuously moist cured until tested. The low resistance shown by the combination containing the non-air-entraining portland cement and natural cement (No. 7) is in line with the results which were reported for a similar combination in 1947.[4] However, the reversal in trend with respect to the comparative durability of the sawed vs. the molded beams shown by this combination has not been explained.

Referring again to table 4, it will be noted that the reduction in flexural strength after freezing and thawing parallels, in general, the reduction in N^2, thus verifying the assumption that the decrease in the natural frequency of vibration of a concrete specimen as the result of alternate freezing and thawing is a measure of its loss in strength. It should be noted, however, that no direct comparison between loss in flexural strength and dura-

Table 4.—Results of durability tests

Mix No.	Cement combination: Portland cement	Cement combination: Blending material	Type of specimen	Air content (gravimetric)	Reduction in N^2 after freezing and thawing for — cycles: 10	25	50	75	100	125	150	175	200	Durability factor	Reduction in flexural strength[1]
				Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.		Pct.
6	Type II	None	Molded	2.3	9	17	28	30	43	--	--	--	--	29	--------
			Sawed	2.3	9	13	23	22	28	[2]39	--	--	--	36	54 (120)
7	Type II	Natural cement	Molded	2.7	9	18	22	23	30	[2]40	--	--	--	36	--------
			Sawed	2.7	8	14	31	37	--	--	--	--	--	24	53 (75)
8	Type II	Ground slag	Molded	5.7	6	13	14	16	20	26	33	35	39	61	--------
			Sawed	5.7	6	9	14	15	20	25	29	30	35	65	52 (200)
9	Type IIA	None	Molded	6.1	7	13	18	18	23	30	39	40	41	60	--------
			Sawed	6.1	3	5	12	12	15	20	26	27	32	68	46 (200)
10	Type IIA	Natural cement	Molded	4.9	8	14	18	18	21	28	35	35	36	64	--------
			Sawed	4.9	6	8	12	14	15	22	27	30	33	67	54 (200)

[1] Reduction in flexural strength after alternate freezing and thawing expressed as a percentage of the strength of corresponding control specimens stored continuously in moist air and tested at the same time. Values in parentheses are number of cycles at which test was made.

[2] These final readings were at 120 cycles.

[2] *Age-strength relations for air-entrained concrete*, by F. H. Jackson. PUBLIC ROADS, vol. 27, No. 2, June 1952.

[3] *The effect on properties of concrete of natural and portland cement blends*, by A. G. Timms, W. E. Grieb, and Geo. Werner. PUBLIC ROADS, vol. 25, No. 2, Dec. 1947.

[4] See footnote 3.

Table 5.—Summary of test results

Mix No.	Cement combination: Portland cement	Cement combination: Blending material	Air content (gravimetric)	Decrease in strength as compared to strength of base mix (No. 1): In flexure, after— 14 days	90 days	365 days	In compression, after— 14 days	15 days	91 days	366 days	Average[1]	Change in durability factor: Sawed beams	Molded beams	Average
			Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.			
1	Type II	None	2.5	---	---	---	---	---	---	---	---	----	----	----
2	Type II	Natural cement	2.8	4	+4	10	11	12	12	18	9	----	----	----
3	Type II	Ground slag	5.4	1	+1	8	13	10	4	11	7	----	----	----
4	Type IIA	None	5.6	8	7	16	10	17	6	23	12	----	----	----
5	Type IIA	Natural cement	4.4	14	1	18	17	25	16	19	16	----	----	----
6	Type II	None	2.3	---	---	---	---	---	---	---	---	----	----	----
7	Type II	Natural cement	2.7	---	---	---	---	---	---	---	---	−12	+7	−2
8	Type II	Ground slag	5.7	---	---	---	---	---	---	---	---	+29	+32	+30
9	Type IIA	None	6.1	---	---	---	---	---	---	---	---	+32	+31	+32
10	Type IIA	Natural cement	4.9	---	---	---	---	---	---	---	---	+31	+35	+33

[1] Average of flexure and compression values.

bility factor is possible due to the variable number of cycles of freezing and thawing at which the strength tests were made. On the other hand, comparison can be made between the loss in flexural strength and the loss in N^2 after the same number of cycles of freezing and thawing. For example, the 54-percent loss in flexural strength for the base mix (No. 6) at 120 cycles can be compared to the drop in N^2 of 39 percent at the end of the same number of cycles. For the several combinations, the ratio of the loss in strength to the drop in N^2 varied from 1.4 to 1.6. Previous investigations have shown that this ratio is usually 1.2 or greater.

A Comparison of Rapid Methods for the Determination of Liquid Limits of Soils

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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Engineers responsible for the routine testing of large numbers of soil samples for highway construction and maintenance need rapid methods for testing. In this study, two simplified methods of determining the liquid limit of soil are compared with the standard method to indicate their reliability and accuracy. Both are "one-run" methods, that is, they require a single determination of moisture and number of blows for groove closure to calculate the liquid limit.

The methods discussed—the chart method developed by the Washington State Department of Highways and a modification of the chart method involving use of a slide rule—have been compared for a wide range of soils found in many of the States. Both of these methods are rapid and reduce the time normally required for testing by as much as 30 to 70 percent. They were found to have good reproducibility and to have sufficient accuracy, within limits, to be acceptable as alternate test procedures for the standard method used for the determination of the liquid limit soil constant.

THE lower liquid limit, developed by Albert Atterberg, was suggested in 1926 as one of the soil eonstants that eould be used to evaluate a soil as a road-building material.[1] This soil eonstant is now widely used by highway and airport engineers to evaluate soils for use as subgrade, base eourse, and embankment material. High liquid limits usually indicate heavy elays and low values indieate friable soils, silts, and elayey sands.

The lower liquid limit, now known as the liquid limit, is defined as the percentage of moisture, based on the dry weight of soil, at which a soil will just begin to flow when jarred slightly. Aeeording to this definition, soils at their liquid limit have a small but definite shear resistance which can be overcome by the applieation of very little foree. The cohesion of the soil-water mixture at the liquid limit is praetically zero.

The determination of this soil eonstant as originally proposed was made by a hand method [1] in whieh the operator adjusted the moisture content by trial and error until exactly 10 blows closed a standard width groove made in the soil sample. The moisture eontent for this soil sample was taken as the liquid limit of the soil. This method required eonsiderable skill and judgment by the operator to obtain duplicate test results.

As soil testing became more widely used, the need for a more praetical test procedure beeame apparent and a meehanieal device [2] was developed which eliminated personal judgment in estimating the proper intensity of the blows to eause elosure of the groove. Although the liquid limit maehine was found to give good reproducibility of results and has been adopted as the AASHO standard test method,[3] it requires more time than the hand method.

The standard method requires that three random trials be made, each at a different moisture content and number of blows; that the percentage of moisture and number of blows be plotted on semilog paper and a flow curve drawn through the points; and that the liquid limit be taken as the moisture eontent eorresponding to the point where the flow eurve intersects the 25-blow line. This procedure has less probable error than a single determination by the hand method, and the maehine ean be used by operators of less skill; but the additional trials, weighings, and eomputations require considerably more time than that required by skilled operators familiar with the use of the one-point hand method.

It is reasonable to expect that engineers responsible for routine testing of soils would be reluetant to aeeept a longer test proeedure without earefully examining the method to determine if modifieations eould be made which would permit more rapid testing of soils.

One of the first modifieations to be tried was patterned after the hand method, by using the meehanieal liquid-limit deviee to furnish the standard foree and adjusting the so water eonsisteney by trial and error un exaetly 25 blows of the maehine elosed t standard width groove.

This modifieation of the maehine metho usually referred to as the moisture adjustme method, ean be developed into an aeeurat rapid method by skilled operators if calibrat with the standard method, and it ean be us effeetively to reduee the time required for t determination of the liquid limit of a so However, it is diffieult for inexperieneed or le skilled operators to adjust the eonsistenty the soil-water mixture to obtain groove closu at exaetly 25 blows.

Chart Method

A rapid method for determining the liqu limit has been devised by Cooper and Johns of the Washington State Department Highways [4] for the routine testing of soils their laboratory. It is ealled the cha method and is based upon the relation betwe the slopes of flow curves obtained for so with different liquid limit test values. Copp and Johnson observed, in their initial stu of soils taken from various parts of the Sta of Washington, that the flow index [5] eomput from the flow curves obtained by the AASH standard method of test inereased uniform with the liquid limit of the soils. Fro these data, six typieal flow eurves, A– shown in figure 1, were developed.

Cooper and Johnson established the val ity of these six typical eurves by plotting t number of blows and moisture eontents f single points representing random trial ru on eaeh of 27 different soils. In this series tests, the number of blows for groo closure was varied within a range of 7 to 6 The liquid limit was determined for eaeh s by drawing a line through the plotted poi and parallel to the nearest typical flow eurv and reeording, as the liquid limit, the mois ure eontent at the interseetion of this li with the 25-blow line. An example is sho by dashed lines in figure 1.

[1] *Adaptation of Atterberg plasticity tests for subgrade soils*, by A. M. Wintermyer. PUBLIC ROADS, vol. 7, No. 6, Aug. 1926.

[2] *Research on the Atterberg limits of soils*, by Arthur Casagrande. PUBLIC ROADS, vol. 13, No. 8, Oct. 1932.

[3] *Standard method of determining the liquid limit of soils*, AASHO designation T 89–49. Standard Specifications for Highway Materials and Methods of Sampling and Testing, part II, 6th ed., p. 228. Published by the American Association of State Highway Officials. 1950.

[4] *A rapid method of determining the liquid limit of soils*, J. H. Cooper and K. A. Johnson. Materials Laborato Report No. 83, Washington State Department of Highwa March 1950.

[5] Flow index is the range in moisture content represent by the number of blows or shocks in one cycle of the lo rithmic scale of a flow curve.

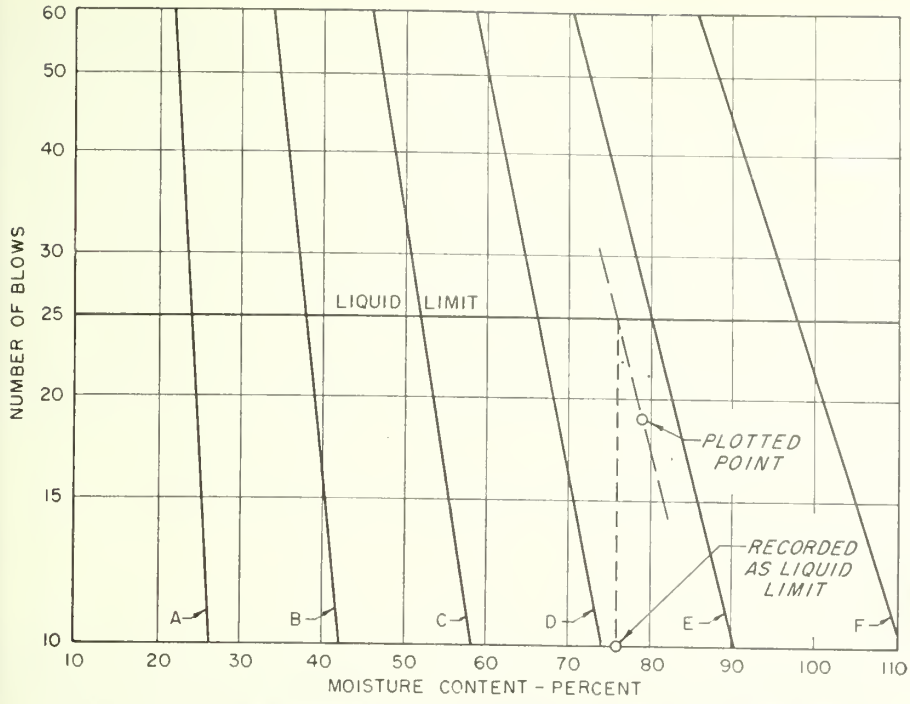


Figure 1.—Relation between slope of typical flow curves and liquid limit.

The values thus obtained from the series of tests were compared with the liquid limits determined by the standard method and good agreement was found between the computed values obtained by this chart method and those determined by the standard test procedure.

Cooper and Johnson made a further simplification of the chart method by subdividing their original six-curve chart to show typical flow curves for each percentage of moisture, so that any point selected from a single moisture content and corresponding number of blows for groove closure could be projected visually to the 25-blow line to obtain the liquid limit.

This improved chart, shown in figure 2, was checked by Cooper and Johnson by using 34 duplicate tests of 73 soil samples. In each case, the liquid limit was determined by the moisture adjustment method and by the chart method. This comparison between the two methods of test showed that discrepancies ranged from 0 to 1.8 percentage points. Because of the low frequency of errors in excess of 1.0 percentage point, however, they considered the accuracy of the chart method adequate for soil classification purposes. They pointed out that two separate closures of the soil-water mixture should be observed to check its consistency before selecting a moisture sample for the determination of the liquid limit by this revised chart method. It was their opinion that the operator should, after completing the second closure, be sufficiently familiar with the flow characteristics of the material to detect any erroneous results.

Although this method is used in their routine identification and classification of soils, Cooper and Johnson recommend that it should not be used for the acceptance or rejection of materials which have borderline values of liquid limit or plasticity index. In these cases, they recommend the use of the AASHO standard mechanical method (designation T 89–49).

The results of a statistical analysis of the basic data used by Cooper and Johnson in the preparation of their original and revised chart methods are shown in table 1. The analysis shows that reduction from the 7–60 to the 15–40 range in the number of blows for groove closure increased the accuracy of the chart method. The maximum deviation in the revised chart method for the 15–40 blow range was 2 percentage points. The deviation was less than 1 point for 88 percent of the tests in the 15–40 blow range, as compared to 70 percent of the tests in the 7–60 blow range.

Table 1.—Comparison of deviations in values of liquid limit obtained by the Cooper and Johnson chart methods from those obtained by standard tests [1]

Range of deviations [2] (percentage points)	Deviation from standard method values of—					
	Six-curve chart method values (groove closure in 7–60 blows)			Multicurve chart method values (groove closure in 15–40 blows)		
	Number of tests deviating in indicated range	Percentage of total number of tests	Cumulative percentage	Number of tests deviating in indicated range	Percentage of total number of tests	Cumulative percentage
0	4	6	6	8	9	9
0.1–0.5	28	38	44	41	48	57
0.6–1.0	19	26	70	26	31	88
1.1–1.5	6	8	78	7	8	96
1.6–2.0	6	8	86	3	4	100
2.1–2.5	6	8	94	--------	--------	--------
2.6–3.0	1	1	95	--------	--------	--------
3.1–3.5	2	3	98	--------	--------	--------
3.6–4.0	0	0	98	--------	--------	--------
over 4.0	2	2	100	--------	--------	--------
Total	74	100	--------	85	100	--------

[1] Duplicate samples tested, one at random blows and the other by the standard test procedure used in the Washington State Department of Highways laboratory.

[2] Deviations ± from the value of liquid limit determined by moisture adjustment method.

Chart Method Investigated

Since only soils from the State of Washington had been studied by Cooper and Johnson, and the moisture adjustment method was the yardstick of comparison, the Bureau of Public Roads widened the scope of the investigation by checking the new method against the AASHO standard method (designation T 89–49) for 364 soil samples selected to represent soils from all sections of the United States. Figure 3 shows the areas from which these samples were obtained. The soils selected had liquid limits ranging from 15 to 104.

The soil test data for checking the accuracy of the chart method were taken from laboratory records and two points with the highest and lowest number of blows were selected from each standard flow curve for each of the 364 soil samples. The moisture contents corresponding to these two numbers of blows for these two points were used for calculating the liquid limit from the revised chart (fig. 2).

To compare the accuracy of these results with those obtained by the standard test method, the data were subdivided according to their liquid limit values into six groups. The first group was terminated at 25 since that is the maximum liquid limit allowed for base-course materials by the AASHO standard specifications, and it is also the upper limit for A–1 soils in the AASHO system of soil classification.[6]

The second group was terminated at 40, the maximum liquid limit for A–4 and A–6 soil groups in the AASHO system of soil classification. The liquid limit values above 40 were arbitrarily subdivided into four 15-point ranges. A tabulation of the deviations of 759 liquid limit values calculated by the chart method from those determined by the AASHO standard mechanical method is shown in table 2. Since the groups do not contain the same number of tests, the frequencies of the deviations, column 4, are expressed in the form of percentages to facilitate comparisons.

[6] *Standard recommended practice for the classification of soils and soil-aggregate mixtures for highway construction purposes*, AASHO designation M 145–49. Standard Specifications for Highway Materials and Methods of Sampling and Testing, part I, 6th ed., p. 29. Published by the American Association of State Highway Officials, 1950.

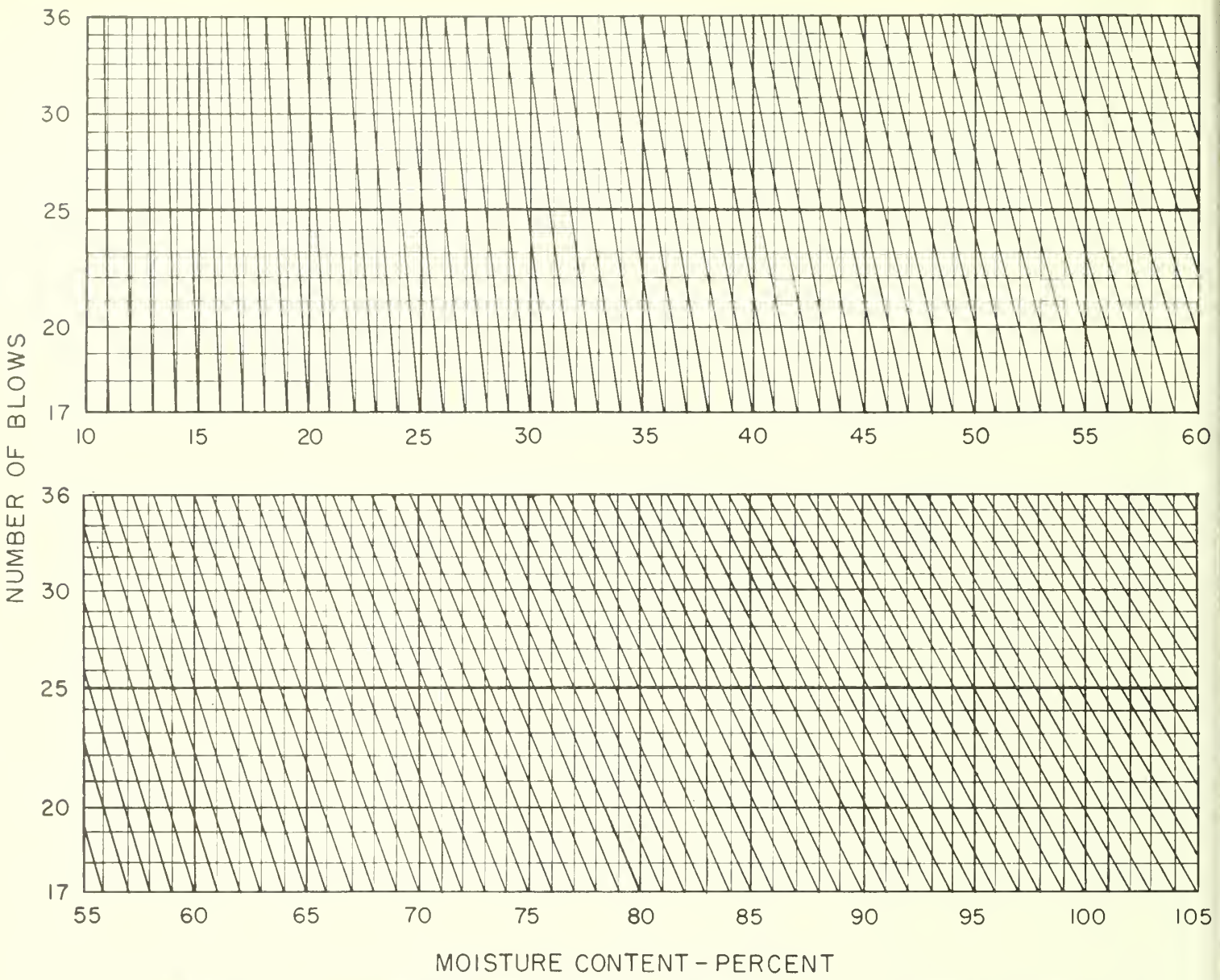


Figure 2.—Multicurve chart developed by Washington State Department of Highways for calculation of the liquid limit.

These data, obtained for 364 soil samples tested by the Bureau of Public Roads, show a range in deviations from the standard values of 0 to about 2 percentage points. This range of deviations compares quite favorably with the range 0 to 1.8 reported by Cooper and Johnson for test data obtained for 73 soil samples taken from the State of Washington.

Slide-Rule Method Developed

The investigation of the accuracy of the chart method for calculation of the liquid limit suggested the possibility of eliminating the use of the chart by the development of a special scale which could be inscribed on a slide rule.

This was accomplished by deriving an equation for the family of typical curves used by Cooper and Johnson in the preparation of their chart. A study of the typical curves in figure 1 indicated a point of convergence near the zero moisture content axis, and while it would be possible to derive an equation for such a family of curves, the problem was simplified by arbitrarily moving the point of convergence to the zero axis. This point is so far from the 25-blow line that the consequent changes in the slopes of the flow curves are negligible within the 17–36 blow range used. The formula derived for this new family of curves is:

$$LL=\frac{W}{1.419-0.3\log S}$$

where

LL = liquid limit.

S = number of blows (or shocks) required to close the groove.

W = percentage of moisture contained by the soil when S blows close the groove.

The use of this formula is facilitated by computing a table of the denominator ($1.419-0.3\log S$) for different values of S, as shown in table 3. The liquid limit for any moisture content at groove closure between 15 and 40 blows is obtained by dividing the percentage of moisture by the value of the denominator in table 3 corresponding to the number of blows required to close the groove.

A more convenient method for using the formula and the data in table 3 is to inscribe a special scale in the space below the B scale of a slide rule to indicate the number of blows required for groove closure corresponding to the values of the denominator of the formula. Figure 4A shows a section of a 20-inch polyphase slide rule with this special scale inscribed for the number of blows between 15 and 40. A study of this special scale with respect to the normal B scale of the slide rule shows that it is constructed by making a mark below the B scale of the slide rule for each value of the denominator of the formula. For example, the 20-blow mark on the special scale corresponds to a B scale reading of 1.029, the 25-blow mark to a value of 1.000 (the midpoint of the B scale), the 30-blow mark to a reading of 0.976, etc.

The calculation of the liquid limit by the slide-rule method is easily made by setting the indicator line over the percentage of moisture found in the soil sample on the A scale of the slide rule and adjusting the slide so that the corresponding number of blows

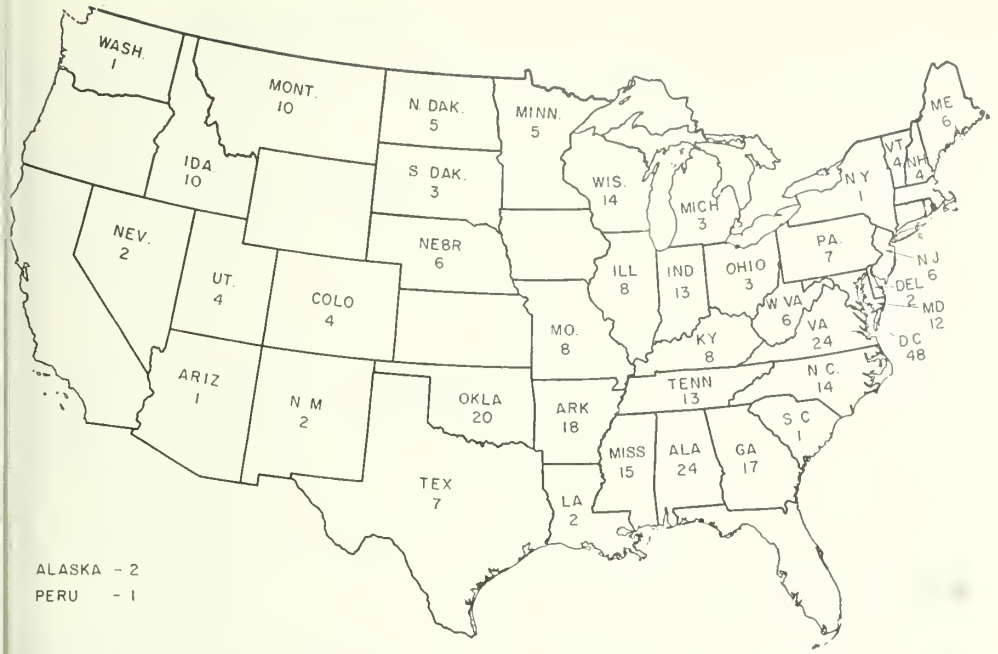


Figure 3.—Number and distribution of soil samples used in testing the relation between the one-point liquid limit tests and the AASHO standard method.

on the special scale coincides with the indicator line. With this setting of the slide rule, the liquid limit is read on the A scale above the end index of the B scale, or on the A scale directly above the 25-blow mark on the special scale.

In the example shown in figure 4B, 20 blows were required to close the groove and the corresponding moisture content was 21.4 percent. The calculated liquid limit, shown on the A scale above the end index of the B scale, is 20.8.

Comparison of Two Methods

The reproducibility of the slide-rule method was checked against the standard method for the same data that were used to check the revised chart method developed by Cooper and Johnson (see table 2, fourth column). The percentage frequency for the same deviations is shown in the last column of table 2, so that comparisons can be made with the chart method. Figure 5 provides a graphic comparison of the accuracy of the slide-rule method and the revised chart method for the ranges of liquid limits selected for this study. The number of tests deviating by ± 0, ± 1, and ± 2 percentage points from the liquid limits determined by the AASHO standard method is expressed on a percentage basis to facilitate the comparison of the accuracy of the two methods.

It is shown by these comparisons that, within the range of liquid limits most generally encountered, the chart and slide-rule methods may deviate from correct values by as much as 2 percentage points. Therefore it is concluded that dependence should not be placed on results obtained by the chart or slide-rule methods for the acceptance or rejection of materials when the liquid limit as determined by either of these methods is within 2 percentage points of the specification limits.

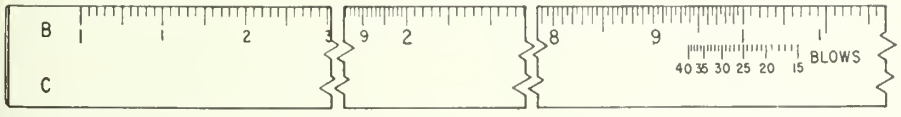
The data in figure 5 show that for the liquid limit range of 15 to 40, over 75 percent of the calculated values for both methods are within ± 1 percentage point and practically all are within ± 2 points of the values determined by the standard test procedure.

However, for the acceptance or rejection of soil-aggregate materials based on a specification having a maximum liquid limit of 25, a deviation of $+0.5$ percentage point from that value, or 2 percent of 25, is the maximum that can be permitted. In order to provide this accuracy, the one-point method needed further improvement. It was thought that this could be accomplished if the range of blows for groove closure was decreased.

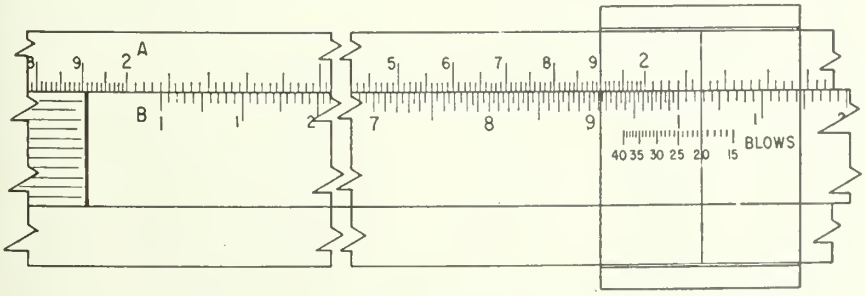
Accuracy of Methods Improved

A statistical analysis was made of the test data to determine the range in number of blows for groove closure which would give results within 2 percent of those obtained by the AASHO standard method. The results of this analysis are shown in table 4 and a study of these data indicates that the desired level of accuracy can be accomplished by narrowing the range of the acceptable number of blows for groove closure from 17–36 to 22–28.

Three ranges in number of blows for groove closure, 29–35, 22–28, and 17–21 are indicated in table 4 to show the reproducibility obtained by chart and slide rule methods for six ranges in liquid limit. The number of tests studied in each of these ranges is also shown. To facilitate comparison of the accuracy of the



A - LOCATION OF SPECIAL SCALE (BLOWS) WITH RESPECT TO B SCALE OF SLIDE RULE



B - SLIDE RULE SET FOR 21.4 PERCENT MOISTURE AT 20 BLOWS, INDICATING CALCULATED LIQUID LIMIT OF 20.8

Figure 4.—Slide rule with special scale for the calculation of liquid limit.

Table 2.—Frequencies of different size deviations of liquid limits derived by chart and slide-rule methods from those obtained by the AASHO standard method

Range of liquid limit	Number of one-point tests	Deviation [1] from AASHO standard	Frequency of deviations of—	
			Chart method	Slide-rule method
		Percentage points	*Percent*	*Percent*
15–25	171	0	76.6	84.8
		1	23.4	15.2
		2	0	0
26–40	209	0	76.1	77.0
		1	24.4	23.0
		2	.5	0
		3	0	0
41–55	129	0	69.0	70.6
		1	27.1	27.9
		2	3.9	1.6
		3	0	0
56–70	82	0	58.6	58.6
		1	40.2	40.2
		2	1.2	1.2
		3	0	0
71–85	94	0	44.7	45.7
		1	47.9	48.9
		2	7.4	5.3
		3	0	0
86–100+	74	0	39.2	35.2
		1	37.8	43.2
		2	20.2	18.9
		3	2.7	2.7
		4	0	0

[1] Deviations from the standard values, originally recorded to the nearest tenth of a percentage point, are here assigned to the nearest whole number.

Table 3.—Values of the denominator of the curve formula corresponding to the number of blows required for groove closure in the liquid limit test

Number of blows	Denominator of formula	Number of blows	Denominator of formula
15	1.066	28	0.985
16	1.059	29	.980
17	1.050	30	.976
18	1.043		
19	1.036	31	.972
20	1.029	32	.968
		33	.964
21	1.023	34	.960
22	1.017	35	.956
23	1.011		
24	1.005	36	.952
25	1.000	37	.948
		38	.945
26	.995	39	.942
27	.990	40	.939

Table 4.—Comparison of accuracy of chart and slide-rule methods [1]

Range of blows for tests	Liquid limit ranges	Number of tests	Frequency of deviations from standard test values (in cumulative percentages)												
			Chart method deviations [2] (±) of							Slide-rule method deviations [2] (±) of						
			0.3−	0.5−	0.8−	1.1−	1.4−	1.7−	2.0+	0.3−	0.5−	0.8−	1.1−	1.4−	1.7−	2.0+
29–35	15–25	31	**42**	74	90	100	100	100	100	**81**	**97**	100	100	100	100	100
	26–40	37	51	84	**95**	100	100	100	100	65	87	**100**	100	100	100	100
	41–55	37	38	54	**92**	**97**	100	100	100	43	73	**95**	**100**	100	100	100
	56–70	22	32	41	86	**96**	**96**	100	100	23	50	86	**100**	**100**	100	100
	71–85	31	23	29	61	81	**90**	**97**	100	23	36	74	87	**97**	**97**	100
	86–100+	20	15	25	30	40	65	**95**	**100**	20	25	35	50	70	**85**	**100**
22–28	15–25	34	88	**100**	100	100	100	100	100	**91**	**100**	100	100	100	100	100
	26–40	27	85	**100**	**100**	100	100	100	100	93	**100**	**100**	100	100	100	100
	41–55	23	65	91	**100**	**100**	100	100	100	65	96	**100**	**100**	100	100	100
	56–70	13	62	92	100	**100**	**100**	100	100	69	92	100	**100**	**100**	100	100
	71–85	28	68	82	96	96	**100**	**100**	100	82	96	100	100	**100**	**100**	100
	86–100+	20	30	45	85	85	95	**100**	**100**	35	45	70	85	95	**100**	**100**
17–21	15–25	37	**51**	**81**	97	97	100	100	100	73	**95**	97	100	100	100	100
	26–40	43	56	77	**100**	100	100	100	100	58	84	98	100	100	100	100
	41–55	25	52	80	88	**96**	100	100	100	40	68	84	88	96	100	100
	56–70	22	36	50	73	**91**	**100**	100	100	36	50	68	**82**	**91**	100	100
	71–85	29	38	41	66	86	**93**	**93**	100	38	59	69	90	**93**	**93**	100
	86–100+	22	9	23	41	50	64	73	**100**	9	18	36	46	55	**59**	**100**

[1] Pairs of boldface values are the percentages of tests with deviations from standard test values less than 2 percent of the minimum and maximum liquid limit ranges shown.
[2] Deviations in percentage points.

methods for each of the three ranges in blows for groove closure, the number of cases having less than the indicated deviations is expressed on a percentage basis for each of the six ranges in liquid limit.

Within each of the ranges of liquid limit, the number of cases having less than each consecutive larger deviation is expressed on a cumulative percentage basis. For example, in the first line of table 4, under the chart method, for the 29–35 blow range and with a liquid limit range of 15–25, the percentages of test results deviating less than 0.3, 0.5, 0.8, 1.1, etc., percentage points from the true values are 42, 74, 90, 100, etc., respectively. The boldface values 42 and 74 are the percentages of test results deviating less than 0.3 and 0.5 percentage points, and are therefore within 2 percent of the maximum and minimum values of liquid limit range of 15–25. Similarly, other comparative values for any designated range in number of blows or liquid limits can be determined for either the chart or slide-rule methods.

A study of these comparative data in table 4 clearly shows that the greatest degree of accuracy for both methods is obtained in the 22–28 blow range and the least accuracy occurs in the 17–21 blow range. Within the 22–28 blow range there are no test values (see boldface results in table 4) calculated by either the chart or the slide-rule methods which exceed the maximum deviations based upon 2-percent limit of error in the liquid limit values determined by the AASHO standard method. The only exception was the deviation of less than 0.3 (which is 2 percent of the minimum value 15 in the 15–25 liquid limit range) which shows 88 percent for the chart method and 91 percent for the slide-rule method. This is insignificant, however, since it is the usual practice to report the liquid limit value to the nearest whole number. For this reason, the 22–28 blow range was selected as the limit to be used to obtain test results meeting the tolerance specified.

Acceptability of Methods

Thus, on the basis of the data obtained in this investigation, both the chart and slide-rule methods should be acceptable as alternates for the standard methods provided (1) that procedure for the preparation of the soil pat for test be that prescribed for the AASHO standard method, (2) that the acceptable number of blows for groove closure be within the 22–28 blow range, and (3) that at least two consecutive closures be observed before selecting the moisture sample for calculation of the liquid limit.

Under these conditions, the results of the slide-rule method have been consistently checked with those obtained by the AASHO standard method in the Bureau of Public Roads laboratory for more than 2 years. There appears to be no difficulty for technicians in the adjustment of the consistency within the 22–28 blow range.

It is suggested that other laboratories make similar comparative check studies, using local soils, to determine whether the accuracy of this improved one-point method of calculation can be established for a range of conditions even wider than those included in this study. If comparable accuracy is obtained by these proposed check tests, it would warrant the use of this method as a standard AASHO and ASTM method.

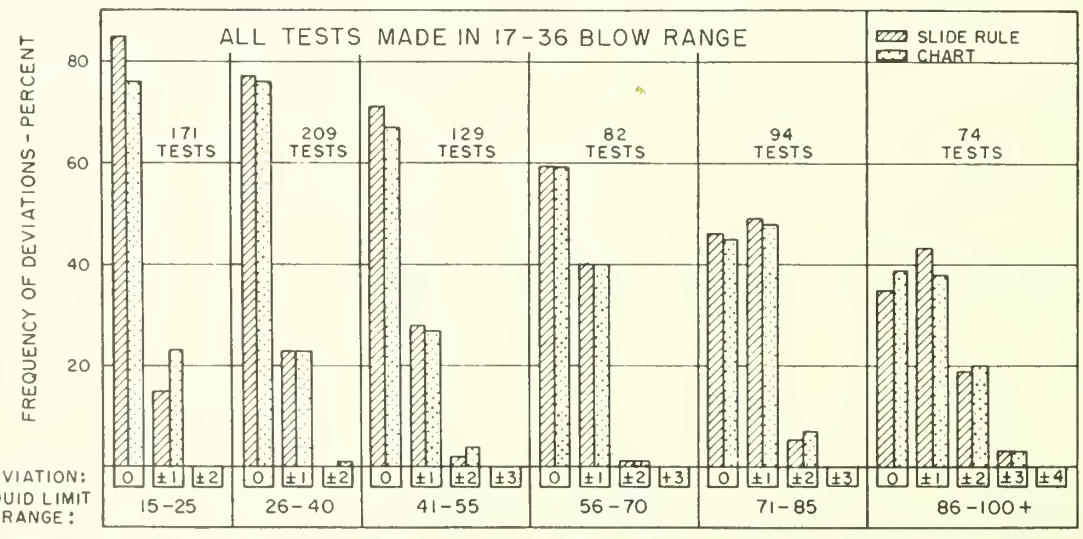


Figure 5.—Percentage of chart and slide-rule tests deviating from the AASHO standard liquid limit.

A Study of Dispersing Agents for Particle-Size Analysis of Soils

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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In the particle-size analysis of soils, mechanical agitation and the physico-chemical action of a dispersing agent are employed to disperse a soil sample in water and prepare a suspension for subsequent determination of particle size-groups by sedimentation techniques. The dispersing agent fulfills two functions vital to the accuracy of the analysis: it assists the mechanical treatment in the initial separation of naturally cemented or aggregated prime particles, and it provides conditions for normal sedimentation in the suspension during the test period by preventing flocculation (or other interfering effects) of the dispersed particles.

Since their introduction to analytical procedures a few decades ago, many different types of chemical dispersing agents have been employed, with varying results. At present, sodium metasilicate is widely used in soil testing for engineering purposes, and sodium hexametaphosphate, sodium hydroxide, and sodium carbonate are among those used for agricultural soils. However, the reported results of numerous studies do not afford a clear evaluation of effectiveness because of the lack of consistency in methods and techniques and, frequently, the narrow scope of experimental soils and dispersants. The investigation reported here was undertaken to provide this needed evaluation.

A large group of dispersing agents representing a wide range of chemical properties were tested with a broad selection of soils under closely controlled conditions of analysis. The resulting data showed sodium metasilicate to be relatively ineffectual in the dispersion of several soils and not suitable for general use. Several others, such as the carbonate, hydroxide, and oxalate of sodium, produced a high degree of dispersion with a few soils, but failed to stabilize the suspensions of others. Sodium polyphos and sodium tripolyphosphate were highly and about equally effective for all of the experimental soils representing many of the great soil groups of continental United States. Two others, sodium hexametaphosphate and sodium tetraphosphate, were only slightly less so. However, none of these four phosphates was effective with laterite, a soil of the tropics. Conversely, this soil was highly dispersed by trisodium phosphate and tetrasodium pyrophosphate, which were generally ineffective with members of the other soil groups.

PARTICLE-SIZE or mechanical analysis is one of the oldest and most valuable test methods applied to soil materials. Its purpose is to determine the quantitative distribution of prime particles in several selected size-groups or separates, such as sand, silt, and clay. The resulting particle-size distribution data have long been used to characterize the physical makeup or texture of soils and have been especially important in soil classification for highway and other purposes.

Although details of analytical procedures vary considerably among different investigators, a required feature of all methods for particle-size analysis is the dispersion of the soil sample. A satisfactory dispersion requires complete separation of the prime particles of the soil or the reduction of agglomerates to sizes not larger than those of the smallest size-group to be determined, and maintenance of the resulting soil suspension in a condition satisfactory for subsequent particle-size determinations. In methods now in general use, dispersion is accomplished by mechanical agitation of a soil-water mixture, in conjunction with an added chemical dispersing agent. Silt- and clay-size groups are determined from the resulting soil-water suspension by sedimentation techniques involving either hydrometer readings or pipetting and drying of aliquots. Sand groups are determined by standard sieving techniques.

The accuracy of the analysis is strongly affected by the degree to which the mechanical agitation and the dispersing agent satisfy the above two requirements. Mechanical dispersion is accomplished by stirring or shaking the soil-water mixture and, for this purpose, a high-speed motor-driven stirrer has been widely used in soil laboratories for many years. A more recently developed air-jet device (*12*)[1] has been shown to produce an even more complete dispersion and also to minimize the attrition of prime particles which may occur with even a brief treatment with the motor-driven stirrer. The use of either device is now optional in AASHO method T 88–49 (*1*).

Purpose of Investigation

The investigation reported here was primarily concerned with the second requirement, or chemical phase, which has been far less well resolved. This concerns the physico-chemical aetivity of the dispersing agent in the soil-water mixture, which assists in the initial mechanical dispersion but is more particularly required for the maintenance of the resulting suspension. The problems involved in this part of the dispersion process result from inorganic and organic substances of clay and colloidal sizes. These adhere to and bind each other, as well as some of the larger particles, more or less firmly into aggregate structures which resist separation. In most highway soil materials, the clay and colloidal particles are largely inorganic in nature and consist chiefly of minute crystallites of one or more members of the clay mineral group. The crystal structures contain an excess of negative charges which attract positively charged ions such as H^+, Ca^{++}, and Na^+. These, in turn, influence the tendency of the clay particles either to be dispersed or to remain in aggregates or floceules. The hydrogen and calcium forms of clays resist dispersion, whereas the particles of sodium clays are separated with relative ease.

Further, a dispersed suspension of soil particles in water may manifest colloidal properties such as gelation (syneresis) or coagulation and flocculation. Coagulation and flocculation hasten particle sedimentation, and gelation retards it. In the first case, hydrometer readings made in the soil suspension for particle-size measurements yield low results for the finer particles, and the analysis makes the soil appear coarser than it actually is; in the second, the reverse is true.

Most dispersing agents provide sodium ions to replace the flocculating cations from the

[1] Italic numbers in parentheses refer to the list of references on page 62.

soil clay, and anions such as hydroxyl, oxalate, carbonate, phosphate, or silicate to inactivate the replaced cations. Or, if instead of an electrolyte, a lyocratic (protective) colloid such as gum acacia is used, dispersion may be aided and the suspension maintained by the action of the colloid in forming protective layers around the clay particles. As often happens in practice, the dispersing agent may fail to provide these effects, with the result that particle sedimentation is interfered with and measurements of the silt and clay fractions are invalidated.

In the long history of particle-size analysis, many different chemical dispersants have been used with varying success and acceptance. Among the more common are ammonium hydroxide and the carbonate, oxalate, and hydroxide of sodium. Sodium hexametaphosphate is now widely used with agricultural soils (*7*, *11*) and AASHO method T 88–49 (*1*) specifies sodium metasilicate. The extensive literature reveals the extreme variability under which analyses have been made, the variations including experimental procedures and apparatus, types and concentrations of electrolytes, types and amounts of soils, and pretreatments of samples for removal of organic matter, soluble salts, or exchangeable bases. Rarely are comparative data presented for more than one or two dispersing agents, and the soils studied are usually severely limited in number and variety. Thus, for the most part, the literature reveals an inadequate testing and evaluation of dispersing agents with regard to degree of dispersion obtained and to suitability for use with a wide variety of soils. It can only be concluded that, in general, the several dispersing agents used in the past vary considerably in both respects. Some are of severely limited value and none has been found suitable for use with all soils.

Summary and Conclusions

This report presents the principal findings of a study designed to evaluate the effectiveness of a large group of dispersing agents with a wide variety of soils under closely controlled conditions of analysis. For purposes of comparison, data obtained by the use of AASHO method T 88–49 are also presented. The criterion adopted for the degree of dispersion obtained is the percentage value for the size-group of particles having a diameter of 1 micron (0.001 mm.) or less. This is determined from the 24-hour hydrometer reading and is termed the "dispersion value." This size-group was selected because it is the smallest now determined in particle-size analyses for highway purposes. Further, its upper-limit is well below the 2-micron value commonly accepted for the clay fraction.

Nineteen chemical dispersing agents were tested to determine their effectiveness in the dispersion of soils and the stabilization of soil suspensions for particle-size analysis. Dispersions were made in an air-jet dispersion cup and with a high-speed motor stirrer. In all, 14 soils and 2 clay minerals were tested. Both soils and dispersing agents were selected to include a wide range of properties and conditions of the respective materials. In all tests, complete particle-size analyses were made and the usual size-groups were determined by the hydrometer method and with standard sieves. Except for variations in kind and amount of dispersant, details of the analyses conformed to the standard AASHO method T 88–49 (*1*). Sodium metasilicate, of the AASHO method, was among the dispersants tested. Principal findings and conclusions from the investigation are as follows:

Thirteen of the dispersing agents were found to be generally unsatisfactory. Several failed to stabilize the suspensions of one or more soils, others frequently produced low dispersion values, and some were ineffective in both respects.

Sodium metasilicate was among the less satisfactory dispersants. Although it failed to stabilize the suspension of only one soil, many of its dispersion values were seriously low. Its use as a general soil dispersing agent should be discouraged.

Widely effective dispersing agents were found only among the phosphate materials. Two of these, sodium polyphos and sodium tripolyphosphate, were highly effective with all soils except one, a laterite. Sodium hexametaphosphate and sodium tetraphosphate were only slightly less effective.

The highest dispersion values for a laterite soil were obtained with trisodium phosphate and tetrasodium pyrophosphate. For most other soils, however, the first was unsatisfactory and the second produced dispersion values that were only moderately high.

From the standpoint of special groupings such as podzolic, illitic, kaolinitic, and montmorillonitic soils, the four phosphates noted in the second paragraph above were consistently highly effective, though their relative ratings varied somewhat among the groups. For general use, sodium polyphos is rated best and sodium tripolyphosphate is a close second. Their unsuitability for the dispersion of a true laterite soil does not extend to related partially laterized soils.

Materials Studied

The 19 dispersing agents selected for this experimental study are listed in table 1, divided among seven subgroups according to chemical composition. These include most of the materials which have been widely used for soil dispersion. A few of the phosphates are new to soil testing, and others such as Lomar PW and Marasperse CB have not been employed with soils but are finding application as industrial dispersing agents.

With a few exceptions, the chemical reagent grade of each material was used. Sodium tripolyphosphate and sodium polyphos were of technical grade and gum acacia and gum ghatti were of pharmaceutical quality. Although the grade of Marasperse CB is not known, its manufacturer has stated that it is a highly purified product. All are readily available at a cost not too high to prohibit their use in routine laboratory work.

The soils chosen for analysis are listed and described in table 2. In addition to the soil names, when known, their liquid and plastic limits and classifications, as derived from the

Table 1.—Experimental dispersing agents

Dispersing agent			Stock solution		Type of reaction
Identification	Name	Formula	Concentration	Grams per liter	
A	Sodium metasilicate (sodium silicate).	$Na_2SiO_3.9H_2O$	0.4N	56.8	Supplies dispersive cation and alkaline reaction. Hydrolyzes to silicic acid which acts as protective colloid (*4*) and precipitates Ca and Mg.
B	Sodium hexametaphosphate [1]	$NaPO_3$ or $(NaPO_3)_6$	0.4N	40.8	Forms soluble undissociated complexes with many cations and prevents flocculation effects. Also supplies a dispersive cation.
C	Trisodium phosphate	$Na_3PO_4.12\ H_2O$	0.4N	50.7	
D	Tetrasodium pyrophosphate	$Na_4P_2O_7.10\ H_2O$	0.4N	44.6	
E	Sodium tripolyphosphate [2]	$Na_5P_3O_{10}$	0.4N	29.4	
F	Sodium tetraphosphate [3]	$Na_6P_4O_{13}$	0.4N	31.3	
G	Sodium polyphos [2]	$Na_{12}P_{10}O_{31}$	0.4N	36.1	
H	Victamide	(4)	0.4N	38.6	
I	Sodium oxalate	$Na_2C_2O_4$	0.4N	26.8	Oxalate ion precipitates Ca and Mg. Supplies dispersive cation.
J	Lithium oxalate	$Li_2C_2O_4$	0.4N	20.4	
K	Ammonium oxalate	$(NH_4)_2C_2O_4.\ H_2O$	0.4N	28.4	
L	Sodium hydroxide	NaOH	0.4N	16.0	Supplies dispersive cation and alkaline reaction.
M	Lithium hydroxide	LiOH	0.4N	9.6	
N	Sodium carbonate	Na_2CO_3	0.4N	21.2	Supplies dispersive cation and alkaline reaction.
O	Lithium carbonate	Li_2CO_3	0.1N	3.7	
P	Gum acacia	Complex colloidal polysaccharide.	0.13%	1.3	Prevents coalescence of particles by providing protective hydration layer.
Q	Gum ghatti	do	0.13%	1.3	
R	Lomar PW	Complex sulfonate [5]	0.4N	95.3	Produces repelling negative charges on suspended particles.
S	Marasperse CB	Complex sulfonate [6]	0.33%	3.3	

[1] Calgon, buffered with Na_2CO_3.

[2] Sodium tripolyphosphate and sodium polyphos are products of the fusion of various combinations of phosphates, purchased from Blockson Chemical Co., Joliet, Ill.

[3] Sodium tetraphosphate is sold under the name of Quadrafos by Rumford Chemical Works, Rumford, N. J.

[4] Victamide is an ammonium amido-polyphosphate furnished by the Victor Chemical Works, Chicago, Ill. Its exact formula has not been released but $(NH_4)_2P_2O_4\ NH.NHP_2O_5(NH_4)_2$ was used to calculate the weight needed for a 0.4N stock solution.

[5] Lomar PW is the sodium salt of a condensed mono-naphthalene sulfonic acid, furnished by Jaques & Co., Passaic, N. J. The formula $(C_{10}H_8SO_3NaCH_2)_2.C_{10}H_8SO_3Na$ was used to calculate the weight needed for a 0.4N stock solution.

[6] Marasperse CB is an indefinite mixture of partially desulfonated sodium lignosulfonates furnished by the Marathon Corp., Rothschild, Wis.

Table 2.—Experimental soils

Soil No.	Series name	Soil group	Geographical source	Liquid limit	Plasticity index	pH (1–1 mlx)	HRB classification and group index
1	Iredell	Yellow podzolic	Fairfax Co., Va	88	62	5.7	A–7–6(20)
2	Cecil	Red podzolic	Lee Co., Ala	63	22	4.7	A–7–5(16)
3	Tenn. I [1]	Red podzolic	Sevier Co., Tenn	37	8	5.1	A–4(6)
4	Tenn. II [1]	Yellow podzolic	Sevier Co., Tenn	38	9	4.9	A–4(6)
5	Sharkey	Alluvium from podzolic materials	Knox Co., Ind	47	24	6.7	A–7–6(15)
6	Nipe	Laterite	Mayagues, P. R	44	13	4.9	A–7–5(10)
7	Miami	Gray-brown podzolic	Tippecanoe Co., Ind	33	18	7.6	A–6(10)
8	Washo [1]	Lake sediment in arid climate	Oneida Co., Idaho	35	9	7.8	A–4(8)
9	Houston	Rendzina (blackland)	Delta Co., Tex	54	32	7.0	A–7–6(19)
10	Yazoo	Alluvium of lower Miss. Valley	Hinds Co., Miss	91	65	7.8	A–7–6(20)
11	Wabash	Alluvium of Miss. Delta	Madison Parish, La	68	40	6.7	A–7–6(20)
12	Salt Flat I [1]	Alkali (high Na_2SO_4)	Lyon Co., Nev	58	35	8.6	A–7–6(20)
13	Salt Flat II [1]	Alkali (high NaCl)	Churchill Co., Nev	23	8	9.4	A–4(8)
14	Putnam	Planosol (clay layer accumulation).	Missouri	50	30	6.5	A–7–6(18)
15	Bentonite	Clay mineral montmorillonite	Wyoming	347	303	8.6	A–7–5(20)
16	Kaolin	Clay mineral kaolinite	Georgia	49	15	8.7	A–7–5(12)

[1] Series name not known; soil occurs in unmapped area.

limit values and the percentages passing the No. 40 and No. 200 sieves, are given as further descriptive information. All of the samples are from the B or C horizons, since this is the part of the soil profile usually used in highway construction.

The soils selected represent a fairly wide range of genetical conditions and include members of many of the great soil groups of the world. Only one of the samples, Nipe (No. 6), is from outside the United States. This is a true laterite, a soil group common to the tropics but not found in this country. Two clay mineral samples, a bentonite and a kaolin (Nos. 15 and 16), were included for testing since they are relatively pure examples of two of the three chief clay mineral groups common to soils. An illite sample was not available in sufficient quantity for the testing of this clay mineral group as such, but illite occurs as a major clay mineral constituent in some of the soils included in the list.

Samples were dispersed (at an air pressure of 20 p.s.i. from a laboratory compressed-air line) with a Model A air-jet dispersion cup which has been described in a previous report (*12*) and approved for use in AASHO method T 88–49 (*1*).

Densities of the soil suspensions were measured with an early model of the streamlined Bouyoucos hydrometer, calibrated for use at 67° F. Density readings were corrected for deviations from this temperature and for the presence of the dispersing agent.

Procedure and Methods

Essentially, the experimental work consisted of the following tests:

1. Soils 1–6 were dispersed in the air-jet dispersion cup with each of 19 dispersants, which were added in 5- or 10-ml. increments of stock solution to replicate portions of each soil.

2. Soils 7–14 and the clay minerals (15 and 16) were dispersed in the air-jet dispersion cup with a single concentration of each of the phosphates.

3. Soils 1–14 were dispersed with the motor stirrer and sodium metasilicate solution.

For those tests made by the use of the air-jet dispersion cup, a 50-g. portion of air-dry soil passing the 2-mm. (No. 10) sieve was soaked overnight in a beaker containing about 75 ml. of distilled water. The mixture was then transferred to the dispersion cup with the use of an additional 75 ml. of water from a wash bottle; the predetermined volume of dispersant stock solution was added; and, with the air-pressure adjusted to 20 p.s.i., the mixture was dispersed for 5 minutes with soils having a plasticity index of 5 and less, 10 minutes with those between 6 and 20, and 15 minutes with those greater than 20. These dispersion times were established previously (*12*) from tests of a large number of soils. The dispersed mixture was then transferred to a 1,000-ml. volumetric cylinder and made up to volume.

In accord with the details of the AASHO method, hydrometer readings were made in the suspension at 2-, 60-, and 1,440-minute intervals for the silt, clay, and "colloid" size-groups and the mixture was then passed through standard sieves for separation of the sand groups. Sediment volumes were recorded at the time of the hydrometer readings and the suspensions were examined visually for evidence of flocculation or other interfering effects. Measurements of pH were obtained for each soil (1–1 soil-water mixture), dispersant solution, and soil-water-dispersant mixture, using a glass electrode and a Beckman Model G pH meter.

For the tests made with the motor stirrer, the mixture was given a 1-minute dispersion in the prescribed cup of this device with 20 ml. of a 3° Baumé (approximately 0.2N) sodium metasilicate solution. Otherwise, the details of the analysis were the same as those given above.

Stock solutions of those dispersing agents having a definite chemical formula were 0.4N, except that of lithium carbonate, which was limited to 0.1N by low solubility. Those of the gums were 0.13 percent and that of Marasperse CB was 0.33 percent, by weight. Weights used for a liter of stock solution are shown in table 1. The volumes used in individual tests ranged from 5 ml. to a maximum at which no further increase in dispersion was measured.

Evaluation of Dispersants

Although percentage values of the usual size-groups were determined in all of the tests, the present report is chiefly concerned with those of the less-than-0.001-mm. size-group, the dispersion values. In the majority of cases in the first series of tests, the dispersion values increased with increasing concentrations of the dispersant, and reached a maximum. With further increases in concentration, varying for different soils and dispersants, the degree of dispersion either persisted at the maximum value or decreased to lower values. Several examples of these effects are illustrated by the curves in figure 1. The concentrations at which maximum values occurred are referred to as the "optimum concentration range." Those yielding values within 1 percentage point of the maximum value are termed the "practical concentration range." Several other effects which also occurred in this series of tests were flocculation at low and high but not at medium concentrations, gelling at certain concentrations, and failure of suspensions over the entire concentration range.

The dispersion values obtained for soils 1–6 at optimum concentrations of each of the dispersants are given in table 3. The figure in parentheses adjoining each dispersion value represents the deviation of that value from the maximum of the 20 dispersion values obtained for that soil. The symbol *F* indicates failure of the suspension due to flocculation or other

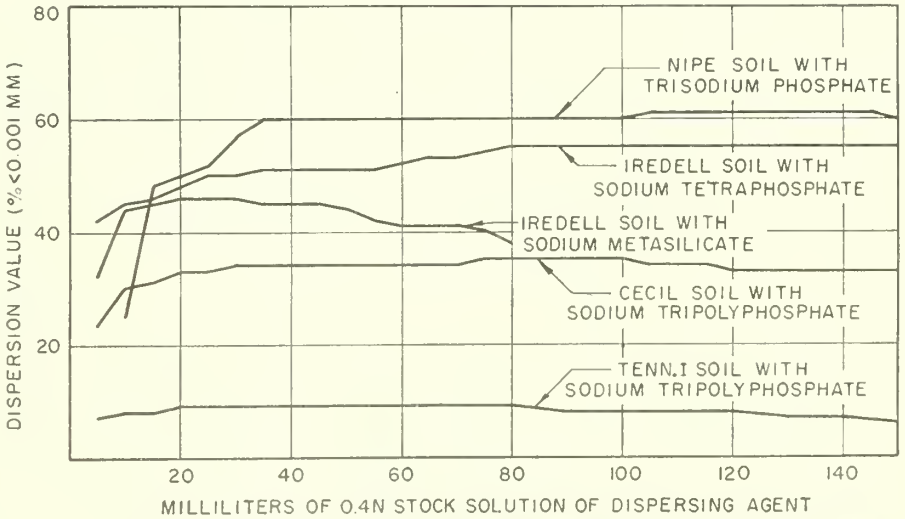

Figure 1.—Effect of dispersing agent concentration on dispersion value.

Table 3.—Effectiveness of 19 dispersing agents with soils Nos. 1–6

Dispersing agent		Dispersion values [1] (percentage of the less-than-0.001-mm. group) obtained with the air-jet dispersion cup at the optimum concentrations of the dispersing agents for soil—						Average deviation from maximum dispersion value (percentage points) for Podzolic soils (Nos. 1–5)
Identification	Name	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	
A	Sodium metasilicate	46 (11)	33 (3)	7 (3)	6 (2)	28 (8)	51 (10)	5.4
B	Sodium hexametaphosphate	55 (2)	36 (0)	10 (0)	7 (1)	34 (2)	52 (9)	1.0
C	Trisodium phosphate	52 (5)	*F*	*F*	*F*	30 (6)	61 (0)	*F*
D	Tetrasodium pyrophosphate	53 (4)	34 (2)	8 (2)	5 (3)	33 (3)	60 (1)	2.8
E	Sodium tripolyphosphate	54 (3)	35 (1)	9 (1)	7 (1)	35 (1)	56 (5)	1.4
F	Sodium tetraphosphate	55 (2)	36 (0)	10 (0)	8 (0)	33 (3)	52 (9)	1.0
G	Sodium polyphos	56 (1)	33 (3)	8 (2)	6 (2)	34 (2)	48 (13)	2.0
H	Victamide	54 (3)	34 (2)	*ND*	*ND*	*ND*	42 (19)	2.5
I	Sodium oxalate	53 (4)	*F*	*F*	5 (3)	32 (4)	*F*	*F*
J	Lithium oxalate	52 (5)	*F*	*F*	3 (5)	34 (2)	*F*	*F*
K	Ammonium oxalate	49 (8)	*F*	*F*	4 (4)	34 (2)	*F*	*F*
L	Sodium hydroxide	54 (3)	*F*	*F*	4 (4)	34 (2)	*F*	*F*
M	Lithium hydroxide	53 (4)	34 (2)	*F*	3 (5)	36 (0)	59 (2)	*F*
N	Sodium carbonate	50 (7)	*F*	*F*	3 (5)	33 (3)	57 (4)	*F*
O	Lithium carbonate	57 (0)	*F*	*F*	3 (5)	33 (3)	41 (20)	*F*
P	Gum acacia	*F*	16 (20)	5 (5)	4 (4)	21 (15)	*F*	*F*
Q	Gum ghatti	44 (13)	15 (21)	9 (1)	5 (3)	23 (13)	14 (47)	10.2
R	Lomar PW	*F*	*F*	9 (1)	5 (3)	*F*	44 (17)	*F*
S	Marasperse CB	52 (5)	22 (14)	9 (1)	7 (1)	28 (8)	43 (18)	5.8
	AASHO method with sodium metasilicate and 1 min. in motor stirrer.	29 (31)	22 (14)	5 (5)	5 (3)	13 (23)	20 (41)	15.2

[1] Numbers in parentheses are deviations (in percentage points) of the given dispersion value from the maximum value obtained for that soil.
F=suspension failure due to flocculation or other colloidal effect; *ND*=value not determined.

colloidal effects at all concentrations of the dispersant.

As indicated by the *F* symbols, ten of the dispersants failed at all concentrations with one or more soils. Two others, gum ghatti and Marasperse CB, maintained the suspensions of all six soils, but were highly ineffectual, as shown by their deviation values. For at least one soil, however, each of these twelve dispersants produced dispersion values equal to or reasonably near the maximum value. This explains why, despite their failure with certain soils, several of these have been considered satisfactory in laboratories where the soils tested are closely related or of limited variety.

For each soil, wide variations occur among the dispersion values given by the several dispersing agents. This indicates that close agreement (± 1 percentage point) between the values from duplicate tests with the same soil does not insure that a specific dispersing agent has provided a high degree of dispersion since other duplicate samples of the same soil, dispersed with a different agent, will also yield closely agreeing dispersion values, though they are higher or lower in degree.

With the podzolic soils 1–5, the majority of the phosphates provide a combination of high dispersion values (low deviations) and freedom of suspensions from flocculation or other colloidal effects. Of the phosphates, B, E, F, and G were highly effective and D and H were only slightly less so. However, C failed at all concentrations with three of the five soils.

With the laterite soil (No. 6), a high degree of dispersion was obtained from only five of the dispersants, among which were three phosphates: C, D, and E. Trisodium phosphate (C), which was ineffectual with the podzolic soils, gave the highest dispersion value obtained. This indicates that the dispersion of laterites presents a different problem from that of the podzolic soils.

For most soils, sodium metasilicate yielded low dispersion values, at optimum concentrations in the air-jet dispersion cup (line 1, table 3), and at the single concentration with the motor stirrer (bottom line of table 3). It has, however, the advantage of preventing flocculation in tests of all six soils. The dispersion values for soils 2 and 5 also provide data for a valid comparison of the effectiveness of the air-jet dispersion cup and the motor stirrer for mechanical dispersion since the single concentration of sodium metasilicate employed in the AASHO method is within the optimum concentration range for both soils in the air-jet dispersion cup. Dispersion values from the air-jet dispersion cup are higher by 11 percentage points with soil 2, and 15 points with soil 5. For the other soils, values from the air-jet dispersion cup range from 1 to 31 percentage points higher, but concentrations of the dispersant are not equivalent.

Table 4.—Practical concentrations [1] of the phosphates

Dispersing agent		Practical concentration range for soil—					Practical concentration for podzolic soils 1–5
Identification	Name	No. 1	No. 2	No. 3	No. 4	No. 5	
B	Sodium hexametaphosphate	85–150	120–150	105–150	25–150	135–150	140
C	Trisodium phosphate	40–55	*F*	*F*	*F*	20–100	55
D	Tetrasodium pyrophosphate	90–140	5–110	20–100	25–140	60–150	100
E	Sodium tripolyphosphate	25–100	30–115	10–120	15–100	45–150	80
F	Sodium tetraphosphate	75–150	50–150	55–150	75–150	65–150	140
G	Sodium polyphos	10–115	30–150	10–80	10–85	30–150	75
H	Victamide	20–95	10–140	*ND*	*ND*	*ND*	90

[1] Expressed as ml. of stock solution.
F=suspension failure due to flocculation or other colloidal effect; *ND*=value not determined.

Further Studies of Phosphates

In the second series of tests, soils 7–14 and the clay minerals bentonite and kaolin (15 and 16) were dispersed at the "practical concentration" of each of the several phosphates. This concentration, expressed as a volume of stock solution, was determined individually for each phosphate from the volume ranges in tests of soils 1–5 that gave dispersion values within 1 percentage point of the maximum. Practical concentration values and the practical volume ranges from which they were derived are listed in table 4.

The results of the second series of tests, shown in table 5, agree in general with those pertaining to the phosphates in the first series and reveal several points of further information:

Sodium tripolyphosphate and sodium polyphos (E and G) are highly effective for the soils and clay minerals, having six optimum dispersion values (zero deviations) out of a possible ten, maximum deviations of 5 and 3 percentage points, and low average deviations of 1.2 and 1.0.

Sodium hexametaphosphate and sodium tetraphosphate (B and F) are somewhat less effective, as shown by their average deviation values of 2.9 and 2.6. This is undoubtedly related to their higher deviations with the clay minerals: 21 and 10 for bentonite, and 4 in both cases for kaolin. Approximately the same degree of effectiveness is shown by tetrasodium pyrophosphate (D) for most of the soils, but two deviations of 8 points raise its average to 4.4 which approximates its dispersion results with the clay minerals.

Trisodium phosphate and Victamide (C and H) each failed to maintain the suspensions of three soils and was only moderately effective with the others. Victamide, however, produced the optimum dispersion value for kaolin but was ineffective with bentonite, and trisodium phosphate was ineffective with both of the clay minerals.

Sodium metasilicate (A) was even less effective than in the previous series of tests, with a suspension failure with one soil and very high deviation values, varying from 10 to 34, with the others.

Summarizing the results obtained with the clay minerals, bentonite was highly dispersed by only sodium tripolyphosphate and sodium polyphos (E and G). Kaolin was highly dispersed by both of these and Victamide (H), and moderately well by sodium hexametaphosphate, tetrasodium pyrophosphate, and sodium tetraphosphate (B, D, and F). It is significant that only sodium tripolyphosphate

Table 5.—Effectiveness of phosphates with soils Nos. 7–14 and the clay minerals bentonite and kaolinite (Nos. 15 and 16)

Dispersing agent			Dispersion value [1] (percentage of less-than-0.001-mm group) for soil or clay mineral—										Deviation from maximum dispersion value (average for soils 7–14)
Identification	Name	Concentration (ml. of stock solution)	No. 7	No. 8	No. 9	No. 10	No. 11	No. 12	No. 13	No. 14	No. 15	No. 16	
B	Sodium hexametaphosphate	140	25 (3)	20 (3)	40 (3)	57 (3)	40 (5)	44 (4)	28 (2)	38 (0)	63 (21)	35 (4)	2.9
C	Trisodium phosphate	55	25 (3)	18 (5)	39 (4)	*F*	39 (6)	*F*	*F*	38 (0)	*F*	32 (7)	*F*
D	Tetrasodium pyrophosphate	100	26 (2)	15 (8)	39 (4)	56 (4)	41 (4)	40 (8)	27 (3)	36 (2)	78 (6)	35 (4)	4.4
E	Sodium tripolyphosphate	80	28 (0)	23 (0)	43 (0)	58 (2)	42 (3)	43 (5)	30 (0)	38 (0)	82 (2)	39 (0)	1.2
F	Sodium tetraphosphate	140	25 (3)	20 (3)	41 (2)	57 (3)	43 (2)	45 (3)	27 (3)	36 (2)	74 (10)	35 (4)	2.6
G	Sodium polyphos	75	26 (2)	20 (3)	41 (2)	60 (0)	45 (0)	48 (0)	29 (1)	38 (0)	81 (0)	39 (0)	1.0
H	Victamide	90	22 (6)	20 (3)	30 (13)	*F*	*F*	*F*	28 (2)	37 (1)	68 (16)	39 (0)	*F*
AASHO method with sodium metasilicate and 1 min. in motor stirrer		20	8 (20)	7 (16)	9 (34)	34 (26)	14 (31)	*F*	20 (10)	10 (28)	*ND*	*ND*	23.6

[1] All values in parentheses are deviations (in percentage points) of the given dispersion value from the maximum value obtained for that soil. *F*=suspension failure due to flocculation or other colloidal effect; *ND*=value not determined.

nd sodium polyphos (E and G) were highly ffective for both of the clay minerals.

The Five Effective Phosphates

In order to compare the essential results btained by using the most effective phos- hates with all of the experimental soils and ith certain special groupings of soils, some f the data were selected from tables 3 and 5 nd rearranged and summarized in table 6.)ue to their failure with several suspensions, hosphates C and H were not included.

Comparison of the results of tests on all of he soils, 1–14, shows sodium tripolyphosphate E) to be most effective, with an average leviation value of 1.4 percentage points, ollowed closely by sodium polyphos, sodium etraphosphate, sodium hexametaphosphate, nd tetrasodium pyrophosphate (G, F, B, and D) in the order of their average deviation alues. The maximum deviation values show hat each dispersant gives a high deviation vith at least one of the soils of the group.

For the soils in the groups 1–5 and 7–14, rom which the laterite is excluded, the best lispersant is sodium polyphos (G), which has very low average deviation value of 0.6. Sodium tripolyphosphate, sodium tetraphos- hate, and sodium hexametaphosphate (E, F, nd B) follow in the order given. The average leviation of sodium pyrophosphate (D) is 3.6 percentage points, which is significantly igher than that of the others.

For the podzolic soils, sodium tripoly- hosphate, sodium hexametaphosphate, and odium tetraphosphate (E, B, and F) are lmost equal in effectiveness, with very closely greeing average deviation values of 1.2, 1.3, and 1.3. Those of sodium polyphos (G) are slightly higher but probably not significantly so. Although tetrasodium pyrophosphate (D) is again the least effective, it is noteworthy that in this group of soils none of the phosphates produced an inordinately high maximum deviation value.

As previously noted, an illite clay mineral, per se, was not available in sufficient quantity for testing. However, data for soils 5, 7, 10, and 11, which are reported as highly illitic (*2*, *5*), are combined in table 6 for an examination of dispersion results where illite is the principal (in percentage) clay mineral. These soils may also be considered as a group containing mixtures of the three chief clay minerals, since the reported analyses (referred to above) indicate that in addition to illite, each contains about 10 percent of kaolinite and 25 percent of montmorillonite. Sodium polyphos (G) again gave highest dispersion, with an average deviation of 1.0; sodium tripolyphosphate (E) is next, with 1.8 percentage points; and sodium tetraphosphate, sodium hexametaphosphate, and tetrasodium pyrophosphate (F, B, and D) follow as a less effective group. Here, again, maximum deviations are not high.

Since only two examples of kaolinitic soils, Cecil and Nipe (2 and 6), are included in the experimental list and since these also contain considerable amounts of iron oxides (*3*, *6*), the specific influence of kaolinite in soils is not precisely defined by the data. However, deviation values given in table 6 for Cecil soil and kaolin (2 and 16), indicate that the five phosphates listed are relatively highly and about equally effective in dispersing these kaolinitic materials. Generally higher deviations for the laterite (Nipe) may be due to the very high percentage of iron oxides present in this soil. However, the moderately high iron oxide content of Cecil soil has not similarly influenced its dispersion characteristics.

Results of the dispersion of bentonite show little resemblance to those of Iredell, Houston, and Putnam soils (1, 9, and 14) in table 6 which have been reported (*6*, *10*) to be highly montmorillonitic. These three soils are highly dispersed by each of the phosphates, though only sodium tripolyphosphate and sodium polyphos (E and G) are highly effective with bentonite. Tetrasodium pyrophosphate (D) is moderately effective, and sodium hexametaphosphate and sodium tetraphosphate (B and F) are unsatisfactory. All of the phosphates were very highly effective with Putnam (14), a clay-pan soil with a clay fraction consisting largely of beidellite, a species of montmorillonite (*8*).

Other Observations

Several examples of the specific size-groups affected during the later stages of the dispersion process are given in tables 7 and 8. Those in table 7 are of soils dispersed in the air-jet dispersion cup with three dispersant concentrations, yielding low, medium, and high degrees of dispersion. The size-groups sustaining losses when the dispersion is increased are found to vary from one soil to another. Losses in Cecil and Iredell occur chiefly in the 0.05–0.005-mm. and 0.005–0.001-mm. groups, and Nipe sustains losses in all of the groups above the less-than-0.001-mm. size. Although a certain amount of attrition of prime particles undoubtedly occurs with any soil,

Table 6.—Effectiveness of phosphates with all soils tested and with certain groups of soils

Dispersing agent		Deviation of the dispersion values from the maximum (percentage points)																		
		All soils (Nos. 1–14)			All soils except laterite (Nos. 1–5 and 7–14)			Podzolic soils (Nos. 1–5 and 7)			Illitic soils (Nos. 5, 7, 10, 11)			Kaolinitic soils			Kaolinite	Montmorillonitic soils				Montmorillonite (Bentonite)
Identification	Name	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	No. 2	No. 6	Avg.	(No. 16)	No. 1	No. 9	No. 14	Avg.	(No. 15)
B	Sodium hexametaphosphate	9	0	2.5	5	0	2.0	3	0	1.3	5	2	3.2	0	9	4.5	4	2	3	0	1.7	21
D	Tetrasodium pyrophosphate	8	1	3.4	8	2	3.6	4	2	2.7	4	2	3.2	2	1	1.5	4	4	4	2	3.3	6
E	Sodium tripolyphosphate	5	0	1.4	5	0	1.2	3	0	1.2	3	1	1.8	1	5	3.0	0	3	0	0	1.0	2
F	Sodium tetraphosphate	9	0	2.4	3	0	1.8	3	0	1.3	3	2	2.5	0	9	4.5	4	2	2	2	2.0	10
G	Sodium polyphos	13	0	2.1	3	0	.6	3	1	2.0	2	0	1.0	3	13	8.0	0	1	2	0	1.0	0

Table 7.—Effect of low, medium, and high degrees of dispersion (three levels of dispersant concentration) on the amount[1] of specific size-groups

Size group (mm.)	Amount found when dispersant concentration is—			Differences			Amount found when dispersant concentration is—			Differences		
	Low (a)	Medium (b)	High (c)	b−a	c−b	c−a	Low (a)	Medium (b)	High (c)	b−a	c−b	c−a
Cecil Soil (No. 2)												
	Dispersed with sodium tripolyphosphate (a=10 ml.; b=20 ml.; c=60 ml.)						Dispersed with sodium polyphos (a=10 ml.; b=30 ml.; c=100 ml.)					
2–0.42	4	5	4	+1	−1	0	4	4	4	0	0	0
0.42–0.25	8	7	8	−1	+1	0	8	8	8	0	0	0
0.25–0.074	20	20	19	0	−1	−1	19	19	19	0	0	0
0.074–0.05	4	4	3	0	−1	−1	5	5	5	0	0	0
0.05–0.005	25	21	22	−4	+1	−3	24	23	21	−1	−2	−3
0.005–0.001	16	13	9	−3	−4	−7	16	11	10	−5	−1	−6
Less than 0.001	23	30	35	+7	+5	+12	24	30	33	+6	+3	+9
Nipe Soil (Laterite, No. 6)												
	Dispersed with trisodium phosphate (a=50 ml.; b=60 ml.; c=120 ml.)						Dispersed with sodium polyphos (a=10 ml.; b=20 ml.; c=60 ml.)					
2–0.42	6	5	3	−1	−2	−3	13	10	8	−3	−2	−5
0.42–0.25	2	1	1	−1	0	−1	5	5	3	0	−2	−2
0.25–0.074	5	4	3	−1	−1	−2	10	9	6	−1	−3	−4
0.074–0.05	1	1	0	0	−1	−1	4	3	2	−1	−1	−2
0.05–0.005	18	15	15	−3	0	−3	24	21	20	−3	−1	−4
0.005–0.001	16	17	17	+1	0	+1	10	12	13	+2	+1	+3
Less than 0.001	52	57	61	+5	+4	+9	36	40	48	+4	+8	+12
Iredell Soil (No. 1)												
	Dispersed with sodium hexametaphosphate (a=10 ml.; b=40 ml.; c=250 ml.)						Dispersed with tetrasodium pyrophos (a=10 ml.; b=30 ml.; c=240 ml.)					
2–0.42	10	9	9	−1	+1	0	10	10	10	0	0	0
0.42–0.25	2	2	2	0	0	0	2	2	2	0	0	0
0.25–0.074	8	9	9	+1	−1	0	8	8	8	0	0	0
0.074–0.05	2	2	1	0	−1	−1	2	2	1	0	−1	−1
0.05–0.005	19	18	16	−1	−2	−3	19	18	17	−1	−1	−2
0.005–0.001	16	10	8	−6	−2	−8	18	13	9	−5	−4	−9
Less than 0.001	43	50	55	+7	+5	+12	41	47	53	+6	+6	+12

[1] Amount found is given as percentage of total weight of soil; differences are in percentage points.

it is believed that little of this effect is reflected in these data.

Since dispersant concentration is the only variable involved in each set of figures, decreases in the coarser sizes are attributed to the breakdown of true aggregates. When the dispersant concentration is increased, the disaggregation increases until a maximum dispersion is reached at the optimum concentration. Differences in the degree of dispersion of the same soil with different dispersants are accordingly attributable to the greater ability of the better dispersant to disintegrate the aggregate structures and to maintain a condition of deflocculation in the resulting suspension.

The data in table 8 show the effects of the two mechanical dispersion devices on the percentages of the several particle-size groups in two soils, Cecil and Sharkey. The values given are for dispersion treatments of 15 minutes with the air-jet dispersion cup and 1-minute with the motor-driven stirrer, using, for both treatments, 20 ml. of 0.2N sodium metasilicate solution, in accordance with the standard AASHO method T 88–49. This amount of the silicate solution also corresponds to the optimum concentration for both soils. The degree of dispersion produced by the air-jet dispersion cup treatment is much greater than that by the motor-driven stirrer, as indicated by the values of the less-than-0.001-mm. group. In the Cecil soil, the gain of this group is derived from small reductions of all of the coarser groups, whereas most of the reduction in the Sharkey soil occurs in the 0.05-0.005-mm. group.

Although an extensive study of the mineralogical nature and the physical and chemical properties of the difficultly dispersed soil constituents was beyond the scope of the present investigation, studies of this sort offer interesting possibilities. It would be of value to know the identity and quantity of the clay or other active fine material which is "bound" in difficultly dispersed aggregates of silt and sand sizes; whether physical properties such as plasticity and volume change of the aggregated clay material are inhibited, and if so, to what extent the properties of the whole soil are affected; and the basic nature of the binding forces in inorganic soils and why and to what extent they are effective in soils of different origin.

Sediment Volumes and pH Values

During the sedimentation of the soil suspensions, a large number of measurements of sediment volumes and pH values (indicating degree of acidity) and observations of colloidal activities were made. Since the measurements do not constitute a thorough and specific study of any of these effects, the bulk of the data is therefore not included in this report. However, some of the findings warrant a brief mention.

Sediment volumes measured in suspensions (after a 24-hour sedimentation period) pre-

Table 8.—Comparison of percentages of various size-groups from dispersions with the air-jet dispersion cup and the motor stirrer [1]

Size-group (mm.)	Amount found (percent of total weight of soil)					
	Cecil soil (No. 2)			Sharkey soil (No. 5)		
	Air-jet dispersion cup	Motor stirrer	Difference [2]	Air-jet dispersion cup	Motor stirrer	Difference [2]
2–0.42	8	9	−1	6	6	0
0.42–0.25	4	6	−2	3	3	0
0.25–0.074	17	20	−3	14	16	−2
0.074–0.05	4	5	−1	3	6	−3
0.05–0.005	23	24	−1	26	35	−9
0.005–0.001	11	14	−3	20	21	−1
Less than 0.001	33	22	+11	28	13	+15

[1] Using the optimum concentration of sodium metasilicate (20 ml. of 0.2N solution).
[2] Difference in percentage points.

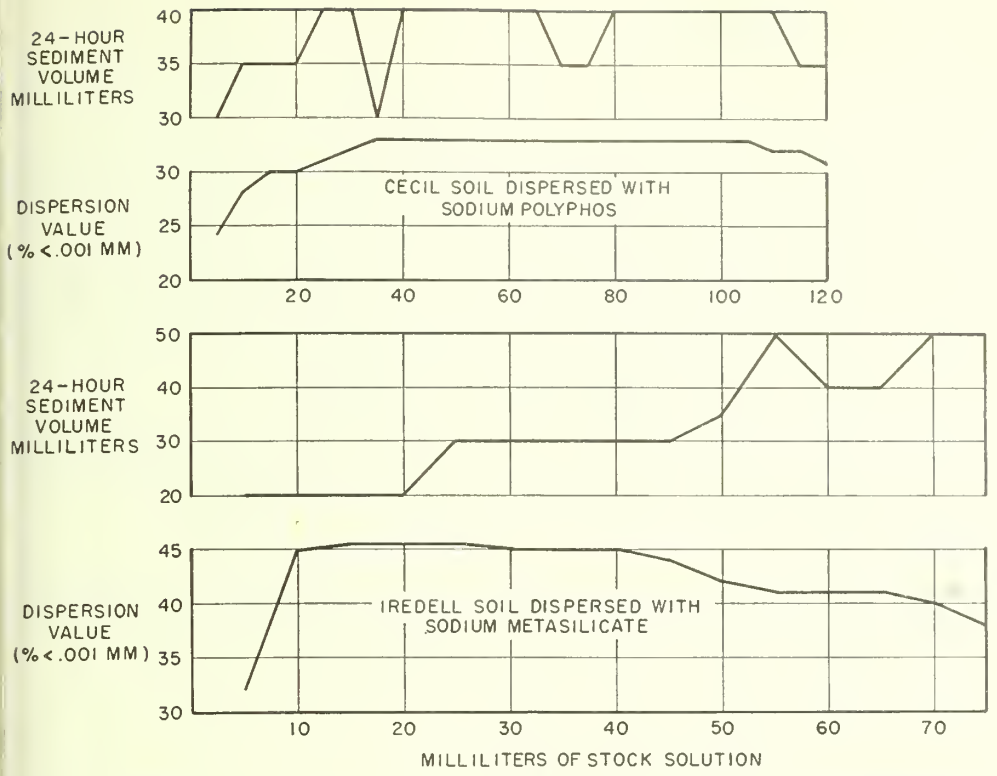
24-HOUR SEDIMENT VOLUME MILLILITERS
DISPERSION VALUE (% < .001 MM)
CECIL SOIL DISPERSED WITH SODIUM POLYPHOS
24-HOUR SEDIMENT VOLUME MILLILITERS
DISPERSION VALUE (% < .001 MM)
IREDELL SOIL DISPERSED WITH SODIUM METASILICATE
MILLILITERS OF STOCK SOLUTION

Figure 2.—Effect of dispersion value on sediment volume with Cecil and Iredell soils.

pared with various concentrations of a dispersing agent are not a good index of the degree of dispersion. In the two examples given in figure 2, contrary to what might be expected, the sediment volume is small at a low degree of dispersion, and increases as dispersion improves. Although the relation varied with different soils and dispersing agents, very similar results were obtained in many of the other tests.

Colloidal effects such as flocculation and gelation, which interfere with the analytical results, are often unnoticed unless they are strongly manifested. However, less evident ones can often be detected by simple tests. The absence of a sharp line of demarcation between the sediment and the suspension, after a brief settling period, invariably indicates an abnormality of the deflocculation process. This observation can be made long before any pronounced flocculation or gelation occurs. A gelled volume in the lower part of the suspension is sometimes observed as a dark layer, but is often obscured by the color of the soil. It can more readily be detected by the pearly iridescence which results when the suspension is caused to rotate in the cylinder by means of a glass rod. With normal deflocculation, stirring produces a pearly effect of light and dark streaks throughout the suspension from the reflection of light by the flat sides of platy particles which are highly oriented by the rotation. If gelation is present in any part of the suspension, this phenomenon is reduced or obscured in the affected area. Gelation is also indicated when, after a reading has been made and the hydrometer removed from the suspension, the liquid remaining on the hydrometer flows off as a film; or when, upon pouring the suspension into a No. 200 or No. 270 sieve for separation and washing of the coarse soil fractions, a restriction of the flow of the suspension through the sieve is noted. Although any of these qualitative tests may be used to advantage, the analytical procedure should include as a minimum requirement a routine examination of each suspension under bright illumination so that the appearance of the demarcation line and any pearly iridescent effect can be observed.

Some of the data from pH measurements of suspensions obtained by the dispersion of each of soils 1–6 with each of the phosphates are given in table 9. The two pH values shown for each soil-dispersant combination are those which occurred at the high and low extremes of the practical concentration range of the dispersing agent when used with that soil. The figure in parentheses adjacent to each pH range is the deviation (in percentage points) of the dispersion value obtained in that range from the maximum obtained for the soil.

As in the previous discussions, the data for soils 1–5 are considered separately from those for soil 6, the laterite. With the group of soils 1–5, the pH range in which each dispersing agent is most effective, is different for different soils. For example, the pH ranges for dispersant B are within narrow limits (6.6 to 7.5) and are nearly identical; those for dispersant E consist of somewhat higher values and are broader in scope (6.8 to 9.1). It is further observed that for any one of the soils of this group, both a high degree of dispersion and good stability of the suspension can occur at several different pH ranges, depending on the dispersant used. However, suspension failure (flocculation of soils 2, 3, and 4 with dispersant C) can also occur over very broad pH ranges which include values at which suspensions are successfully maintained by other dispersants. The stock solutions of the better dispersants, B, E, F, and G, for soils 1–5 have pH values between 6.7 and 8.8; that of D, which is less effective, is 10.0; and those of C and H, which are least effective, are 11.3 and 4.3

In contrast to soils 1–5, the laterite (6) is effectively dispersed only under conditions of high pH; its deviations are lowest (best dispersion) at pH values of 9.2 and above with dispersants C and D, intermediate in the range 8.1–9.0 with dispersant E, and highest (poorest dispersion) in the range 6.2–6.7 with dispersant H.

Puri has presented data (*9*) showing that many soils are most highly dispersed at a pH of 11.2 when sodium hydroxide is used as the dispersant and the soil samples are pretreated to convert them to the H-form. The data in table 9 are at variance with Puri's findings, since they were obtained using soil samples in their natural cationic state, and phosphate materials which are more effective at lower pH values. The present investigation was concerned with dispersion techniques which provide a high degree of dispersion without time-consuming pretreatments of the soil samples.

Table 9.—pH values in suspensions of soils 1–6 with the phosphates

Dispersing agent		pH ranges [1] and deviations [2] from the maximum dispersion values for soil —					
Identification	pH of stock solution	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
B	6.8	6.6–7.0 (2)	7.3– 7.3 (0)	7.4– 7.4 (0)	7.1– 7.2 (1)	7.2– 7.5 (2)	7.4– 7.4 (9)
C	11.3	8.7–9.2 (5)	6.7–10.7 *F*	7.6–11.1 *F*	7.2–11.4 *F*	9.4–10.9 (6)	9.5–11.3 (0)
D	10.0	8.2–8.7 (4)	6.2– 9.0 (2)	8.6– 9.5 (2)	8.5– 9.3 (3)	8.8– 9.3 (3)	9.2– 9.4 (1)
E	8.8	7.2–7.6 (3)	6.8– 8.4 (1)	7.3– 9.1 (1)	7.2– 8.9 (1)	7.9– 8.8 (1)	8.1– 9.0 (5)
F	7.6	6.7–7.2 (2)	7.8– 8.1 (0)	8.1– 8.2 (0)	7.3– 8.1 (0)	7.4– 7.8 (3)	7.9– 8.1 (9)
G	6.7	6.6–6.8 (1)	5.8– 6.8 (3)	7.2– 7.3 (2)	6.6– 7.3 (2)	7.0– 7.3 (2)	7.3– 7.4 (13)
H	4.3	5.6–6.1 (3)	6.0– 6.2 (2)	*ND*	*ND*	*ND*	6.2– 6.7 (19)
pH of soil (1–1 mix)		5.7	4.6	5.1	4.9	6.7	4.9

[1] The two values shown for each soil-dispersant combination are those which occurred at the high and low extremes of the practical concentration range of the dispersing agent when used with that soil.

[2] All values in parentheses are deviations (in percentage points) of the given dispersion value from the maximum value obtained for that soil.

F=flocculation at all concentrations and over the pH range cited; *ND*=pH not determined.

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1946, 50 cents. 1949, 55 cents. 1952, 75 cents.
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Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

Motor-Vehicle Traffic Conditions in the United States, House Document No. 462 (1938):
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Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

Public Control of Highway Access and Roadside Development (1947). 35 cents.

Public Land Acquisition for Highway Purposes (1943). 10 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

Roadside Improvement, No. 191MP (1934). 10 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.

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- Act III.—Uniform Motor-Vehicle Civil Liability Act (1944). 10 cents.
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- Act V.—Uniform Act Regulating Traffic on Highways (revised 1952). 20 cents.
- Model Traffic Ordinance (revised 1953). 20 cents.

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AS OF JUNE 30, 1954

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$6,079	$9,554	$4,923	310.0	$12,137	$6,268	216.9	$43,978	$21,900	477.2	$65,669	$33,091	1,004.1
Arizona	2,027	5,270	3,681	125.5	1,421	1,058	33.5	6,468	4,518	94.6	13,159	9,257	253.6
Arkansas	3,784	10,172	5,847	409.8	5,754	2,831	171.4	18,220	9,079	411.3	34,146	17,757	992.5
California	1,432	19,038	8,626	159.3	26,413	11,980	77.8	97,492	46,043	271.0	142,943	68,649	508.1
Colorado	9,201	4,033	2,258	39.2	333	187	1.6	18,629	10,183	206.7	22,995	12,628	247.5
Connecticut	8,968	2,652	1,326	7.9	4,600	2,122	6.1	5,090	2,554	12.6	12,342	6,002	26.6
Delaware	1,566	2,530	1,290	9.8	2,994	1,841	8.1	5,551	2,763	34.3	11,075	5,894	52.2
Florida	4,163	24,173	12,247	389.4	5,428	2,742	62.2	20,863	11,016	332.4	50,464	26,005	784.0
Georgia	13,358	10,134	5,211	196.0	7,073	3,497	141.5	37,823	17,740	545.4	55,030	26,448	882.9
Idaho	1,877	4,662	3,012	90.4	6,629	4,135	97.4	15,796	9,798	287.5	27,067	16,945	484.3
Illinois	12,770	34,465	18,472	345.7	10,779	5,194	121.7	77,294	40,731	485.7	122,038	64,397	953.1
Indiana	7,204	44,193	23,104	184.9	19,956	10,078	97.8	22,168	11,509	143.6	86,317	44,691	426.3
Iowa	4,248	11,172	6,151	395.4	8,964	4,807	380.2	27,654	14,534	847.3	47,090	25,492	1,624.9
Kansas	6,384	10,312	5,200	804.1	6,622	3,325	495.5	19,197	9,677	720.0	36,131	18,202	2,019.9
Kentucky	5,803	9,995	5,31[illegible]	102.6	7,496	3,752	94.0	23,685	12,406	272.3	41,378	21,470	468.9
Louisiana	5,656	15,302	7,640	115.8	15,901	6,890	98.3	24,326	11,755	99.1	55,529	26,285	306.6
Maine	3,960	5,455	2,804	45.4	509	255	5.8	12,985	6,424	97.5	18,949	9,483	148.7
Maryland	6,466	12,976	6,739	81.5	5,097	2,790	61.5	9,242	4,663	43.8	27,315	14,192	186.8
Massachusetts	9,337	12,985	6,133	20.1	1,685	988	.3	57,080	26,025	42.8	71,050	33,946	65.2
Michigan	3,809	39,992	19,995	466.4	16,060	8,034	355.8	40,385	19,199	271.3	96,397	47,228	1,123.5
Minnesota	4,573	10,374	5,373	758.7	12,832	6,668	928.2	25,659	13,349	614.4	48,865	25,390	2,301.3
Mississippi	4,568	11,276	5,721	353.2	6,229	3,198	224.8	22,433	11,290	601.7	39,940	20,209	1,180.3
Missouri	10,060	17,670	9,111	861.4	8,697	4,621	319.4	57,147	28,293	581.1	83,514	42,025	1,761.9
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Nevada	6,260	3,548	2,185	21.6	762	653	44.0	8,732	7,249	236.0	12,062	10,087	361.6
New Hampshire	2,192	4,376	2,188	26.8	1,636	818	6.3	7,150	3,635	38.9	13,162	6,635	72.0
New Jersey	10,191	4,233	2,117	51.7	1,976	949	5.5	23,023	8,856	15.8	29,232	11,922	73.0
New Mexico	2,452	5,264	3,329	129.0	2,484	1,575	42.4	9,848	6,072	227.1	17,596	10,976	398.5
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Ohio	7,308	25,751	13,000	108.9	19,313	8,899	40.0	85,755	39,863	162.2	130,819	61,762	305.1
Oklahoma	7,099	21,116	11,405	321.2	6,234	3,275	149.8	16,523	8,771	234.8	43,873	23,451	705.8
Oregon	2,622	3,409	1,983	23.3	2,019	1,213	73.0	15,726	9,520	209.1	21,154	12,716	305.4
Pennsylvania	3,898	22,255	11,602	16.9	36,127	17,868	56.9	95,817	46,820	214.4	155,199	76,310	288.2
Rhode Island	1,094	8,118	4,059	37.8	994	496	6.9	7,654	3,925	36.5	16,966	8,480	72.8
South Carolina	6,265	8,792	4,701	178.7	3,801	1,993	141.9	16,081	8,171	525.5	28,674	14,865	846.1
South Dakota	1,978	12,627	7,163	653.6	1,853	1,157	117.7	8,869	5,048	460.9	23,349	13,368	1,252.2
Tennessee	7,073	14,438	7,172	372.5	7,431	3,724	201.3	36,654	16,733	494.3	58,523	27,629	1,068.4
Texas	13,658	9,691	5,054	80.0	15,991	7,905	445.9	66,733	35,614	1,121.3	92,415	48,573	1,655.2
Utah	506	5,260	4,057	78.7	3,358	2,509	97.3	9,117	6,978	99.1	17,735	13,544	275.1
Vermont	1,208	2,902	1,459	33.7	1,130	554	7.7	8,945	4,515	65.7	12,977	6,528	107.1
Virginia	4,307	14,635	7,108	249.7	6,318	3,142	129.7	29,318	13,915	103.0	50,271	24,165	562.4
Washington	1,358	12,911	6,767	120.6	7,248	3,627	169.3	19,375	10,275	189.5	39,534	20,669	479.4
West Virginia	6,144	10,006	5,055	56.5	1,976	1,000	3.1	16,243	8,094	41.5	28,225	14,149	101.1
Wisconsin	3,685	17,987	9,471	176.3	8,810	4,366	193.1	27,900	13,809	361.4	54,697	27,666	730.8
Wyoming	544	5,638	3,669	108.5	1,029	667	20.0	9,654	6,050	250.3	16,321	10,426	378.8
Hawaii	3,184	932	466	2.0	634	309	2.2	11,969	5,651	15.5	13,535	6,426	19.7
District of Columbia	4,298	6,793	3,157	2.9	996	498	2.3	10,569	5,081	1.4	18,358	8,736	6.6
Puerto Rico	5,507	6,588	3,124	38.5	5,239	2,182	9.2	13,937	6,589	40.9	25,764	11,895	88.6
TOTAL	261,168	702,021	367,671	11,829.1	365,714	195,442	6,951.4	1,522,413	765,789	15,374.9	2,610,148	1,329,102	34,155.4

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Public Roads

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Asphalt-rubber surfaced street, Washington, D. C.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 28, No. 4 *October 1954*

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Edgar A. Stromberg, *Editor*

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

Contents of this publication may be reprinted. Mention of source is requested.

Two Articles on Rubber-Asphalt:

The Effect of Various Rubbers on the Properties of Petroleum Asphalts

A Laboratory Study of Rubber-Asphalt Paving Mixtures

Public attention has been caught, in recent years, by reports of the experimental use of rubber in bituminous pavements. Claims have been made that the use of rubber in various forms extends the life of bituminous surfaces, increases their elasticity, decreases their brittleness at low temperatures, increases their resistance to the action of water and to abrasion by traffic, reduces skidding, and minimizes maintenance requirements.

Many experimental rubber-asphalt pavement sections have been built to determine under actual service conditions the comparative degree of progressive alteration in properties of the rubber-asphalt mixtures that occurs with age. Skid resistance measurements have also been made. The Bureau of Public Roads has observed the construction of some of these test sections, and is following the study of their performance with close interest.

The bibliography in this issue of PUBLIC ROADS indicates the wide variety of experimental work that has been done in the use of rubber in bituminous pavements. No technical analyses of general coverage of the subject have appeared, however, and for this reason the Bureau of Public Roads undertook the laboratory research reported in the two articles presented here.

The amount of rubber used in a "rubber road" is quite small. The asphalt in a bituminous pavement serves as the cementing agent for the gravel or crushed stone, sand, and mineral filler which make up the bulk of the mixture. When rubber is added, it amounts to only about 5 percent of the asphalt, which itself is only from 5 to 10 percent of the total weight of the pavement material. Thus the rubber represents about one-fourth to one-half of one percent of the weight of the paving mixture.

Nature of Studies

The first article in this issue of PUBLIC ROADS describes a study of rubber-asphalt blends, either prepared in the laboratory or extracted from pavement mixtures sampled in the field. Three asphalts and fourteen different rubbers—natural, synthetic, and reclaimed—were used in the laboratory-prepared blends. Mineral aggregate was not involved in the laboratory tests described in this study.

The second article reports a laboratory study of rubber-asphalt paving mixtures, mineral aggregate being included in this case. The asphalt used was one of the three included in the first study, and is identified as AC-2 in both reports. In the preparation of the mixtures, the rubber was either preblended with the asphalt and the blend then mixed with the aggregate, or the rubber in powder or crumb form was added to the hot aggregate before addition of the asphalt. The plasticized rubber and the three powdered rubber materials identified as R-1, R-2, and R-6 were used in both studies.

Rubber-Asphalt Blends

Disregarding some inconsistencies, the character of the rubber-asphalt blends was affected by the type and amount of rubber used, the nature and source of the asphalt, and the temperature, time of heating, and amount of stirring in the preparation of the blends.

In general, the addition of rubber increased the softening point and viscosity of asphalts and decreased their penetration, flow, susceptibility to temperature change, and ductility at 77° F. With two exceptions, the low-temperature ductility of the asphalts was not very much influenced by the addition of rubber. There was a wide difference in the resistance of different rubber-asphalt blends to the changes produced by high temperatures such as are required in the processing of paving mixtures. Tests with one asphalt showed that its elasticity was increased in varying degree by the addition of various rubbers.

Tests of rubber-asphalt in pavement mixtures taken from experimental road sections were fruitless, since the properties of the rubber-asphalt material cannot be determined by available methods of extraction and recovery.

Rubber-Asphalt Paving Mixtures

In the study of rubber-asphalt paving mixtures, involving mineral aggregate, both favorable and unfavorable results were indicated. When powdered rubber was used, the compactibility of the paving mixture was lowered and there was higher susceptibility to temperature change than in the comparable mixes without rubber. Mixtures with preblended rubber were much more compactible and stable than mixtures in which the rubber was added as a powder. Mixtures containing three of the four preblended rubbers showed higher stabilities than mixtures without rubber.

Tests of the paving mixtures failed to indicate that bituminous surfaces containing rubber would be both substantially more plastic at low temperatures and substantially less plastic at high temperatures than surfaces without rubber. Mixtures containing plasticized rubber or preblended natural rubber were less plastic at both 77° and 140° F. than mixtures with the rubber omitted. In general, mixtures containing synthetic or reclaimed rubber, in either powder or preblend form, were more plastic at both 77° and 140° F. than the control mix without rubber. The single exception was the mixture containing preblended synthetic rubber which, after oven exposure for 21 days, was slightly less plastic at 140° F. and slightly more plastic at 77° F. than the mix without rubber.

The tests showed that the addition of rubber in either powder or preblend form did not increase the resistance of the mixtures to the action of water.

Mixtures of Ottawa sand and preblended asphalt and natural rubber were found to be more resistant to abrasion than mixtures without rubber or any of the other rubber-asphalt mixtures.

Significance of Tests

What do these laboratory tests signify? The effects of the addition of different rubbers to different asphalts may vary widely: the use of rubber shows promise of benefits in some respects; in others it does not. Conclusions as to benefits of real economic value in the addition of rubber to asphalt must wait on further observation of the behavior of experimental pavements under the influence of age, weather, and traffic. None of these experimental pavements have as yet shown significant behavior differences between comparable rubber-asphalt and plain asphalt sections.

312695—54——1

The Effect of Various Rubbers on the Properties of Petroleum Asphalts

BY THE PHYSICAL RESEARCH BRANCH

BUREAU OF PUBLIC ROADS

Reported by RICHARD H. LEWIS, Senior Chemist, and J. YORK WELBORN, Highway Physical Research Engineer

THE investigations of the effect of rubber on bituminous materials (*1*) [1] used in paving mixtures were initially confined to laboratory studies. These investigations were concerned with the changes that occurred when rubber latex was blended with blown, native, and petroleum asphalts and with asphaltic emulsions.

In 1933, an unvulcanized rubber powder, first known as Stam rubber powder and later (after modifications in processing) as Pulvatex, was developed by the Department of Rubber Research of the Experimental Station, West Java, at Buitenzorg, Java. In 1938, Mealorub, formerly called P.W.J. rubber powder or Van Dalfsen rubber powder, was developed by the same research institution. These two materials are protected by patents of the Rubber Foundation at Delft, Netherlands.

Pulvatex is prepared by spraying field latex or creamed latex in a current of hot air, while simultaneously a certain quantity of dried diatomaceous earth is blown in the dispersed particles so that a dehydrated mass is produced in which the dessicated rubber particles do not adhere. Mealorub is prepared by heating fresh latex with certain chemicals by means of which sulfur is fixed. The product is then flocculated by means of acid, dehydrated, disintegrated, and dried. Mealorub has undergone a certain amount of vulcanization, in contrast to the unvulcanized Pulvatex. The two materials are unlike in appearance: Pulvatex has a grayish brown color and the texture of flour; Mealorub is a yellow crumbly material having the appearance of coarsely ground corn meal.

Other rubbers in powdered or crumb form have been developed from natural rubber latex (*2*) but the natural rubber powder generally used in asphaltic construction in the earlier work was of the Mealorub or Pulvatex types.

The results of the investigations covering the effects of both the rubber latex and the powdered or crumb rubbers showed that the consistency, softening point, temperature susceptibility and ductility of the asphalt were so altered that the performance of pavements and surfacing mixtures might be materially benefited by the admixture of rubber compounds.

[1] Italic numbers in parentheses refer to the bibliography on pages 88 and 89.

Test pavements

Experimental sections of surface dressings, composed of sand, filler, cutback asphalt, and a small amount of rubber powder, laid on old asphaltic concrete roads in the Netherlands from 1935 to 1938, were still in excellent condition after being subjected to the heavy traffic imposed by the German and Allied armies during World War II (*3*). The sections sealed with the rubberized surfacing mixture were in much better condition than the untreated asphaltic concrete. However, according to available reports, a similar surfacing mixture without added rubber was not used over the old asphaltic concrete to provide a direct comparison of the surface dressings with and without the rubber admixture.

Nevertheless, the excellent condition of the rubberized asphaltic seal treatments impressed representatives of the rubber industry in the United States who had inspected these roads. As a result, the Goodyear Tire and Rubber Company developed synthetic rubber powder of the GR–S Type V. It was vulcanized in latex form to prevent coalescence on drying. Several experimental sections of asphaltic pavement containing this material were constructed in 1947. On the basis of performance of these sections, the first large-scale construction of an asphaltic pavement containing rubberized asphalt in any form in the United States was laid on Exchange Street in Akron, Ohio, in September 1948 (*4*).

From 1948 to date, asphaltic surfaces containing rubber in some form have been built elsewhere in the United States, and in Canada, Europe, the Far East, Australia, New Zealand, and South Africa (*5*).

In most of the areas outside the United States, natural rubber, similar to Mealorub, has been used with liquid or semisolid asphaltic materials in asphaltic paving mixtures and in surface treatment work. Some construction in the Far East has been done with a rubberized binder prepared by combining rubber latex and emulsified asphalts. In the United States natural, synthetic, reclaimed, processed, and scrap rubber in powdered form have been used in asphaltic paving mixtures, and powdered natural rubber added to cutback asphalts has been used for seal coating and surface treatment work. Blends of synthetic rubber latex and emulsified asphalt have been used as binders for hot asphaltic concrete in one State. Blends of plasticized rubber asphalt, normally used for joint sealing purposes, and meeting the test requirements of Federal specification SS–S–164, and petroleum asphalts have also been used as binders for asphaltic concrete and sheet asphalt mixtures.

Preblended binders used

In May 1950, experimental rubberized asphaltic pavements were constructed in Singapore (*6*), using natural rubber in the amount of 5 percent of the asphalt, incorporated into the mixtures in various ways. In one mixture, the powdered rubber was added to the aggregate at the mixer box. In a second mixture, the powdered rubber was added to the hot asphalt in the asphalt bucket and, after a short period of stirring to distribute the powdered rubber, the contents of the asphalt bucket were dumped over the heated aggregate in the mixer box. In a third mixture, the rubber powder was preblended by heating the asphalt to 330° F., adding the required amount of rubber, and stirring the rubber-asphalt blend for 4 hours at the 330° F. temperature. The blend was then poured onto the hot aggregate in the mixer box, the mineral filler added, and the ingredients mixed for 3 minutes. The temperature of the mixture was about 300° F. when dumped from the mixing box and about 275° F. at the time of laying. No data were presented to show the test characteristics of the two rubberized blends used in the mixtures.

An inspection report on the condition of these experimental sections (*7*) indicated that the sections containing the rubber which was preblended with the asphalt for 4 hours before adding to the hot aggregate were in a satisfactory condition, while the sections in which the rubber was added to hot asphalt for a short mixing period before pouring into the mixer, and those in which the rubber powder was added directly to the hot aggregate, were still deficient in stability 12 months after construction.

In a more recent article (*8*), descriptions are given of several experimental road surfacings, laid during 1950 and 1951 in various parts of the British Empire, in which Mealorub rubber powder was used with asphalt. On the basis of these road trials several methods of adding the rubber to the finished mixture were recommended, with special regard to the temperatures employed. In preblending the rubber with the asphalt before incorporation in

ving mixtures, it was recommended that e powder be added slowly and the blend itated for not less than 1 hour at 325° F. 2 hours at 300° F. It was stated that the eblended rubber-asphalt prepared under the ove conditions could be cooled and again eated for use. It was also stated that if rubber-asphalt blend is stored hot before , it may be maintained at a temperature of 5° F. for 24 hours, 300° F. for 2 days, 280° for 4 days, or 260° F. for 8 days without ious alterations to the properties of the nd; but temperatures above 320° F. should avoided.

t also was recommended that, due to the ditional tenacity imparted by the rubber he mixture, the temperature of the mixture uld be approximately 20° F. higher than t usually used. Again, no test results were en in this report on the preblended rubber-halt mixtures to evaluate the changes that urred in the test characteristics of the halts during their blending and storage.

n 1952, the Massachusetts Department of blic Works resurfaced a 1.7-mile section of S. 6 with standard bituminous concrete . The specifications required that the halt in one-half of the section contain 7.5 cent by weight of GR–S synthetic rubber der, the type of which was not given, ed and dispersed in the asphalt by the owing procedure:

> The specified amount of powder is added at a minimum of 200° F. with suitable agitation. It is sifted through a ¼-inch mesh sieve into the mixture to prevent lumping. The temperature of the mixture is then raised to 250 to 275° F. and the mixing is continued for a minimum of 2 hours. The asphalt containing the dispersed rubber powder is then mixed with the aggregate in the usual manner.

he description of this project the physical haracteristics of the rubberized asphalt l d were not reported.

y thetic preblend in Akron

1 September 1948, the Goodyear Tire and ubber Company, in cooperation with the Ci of Akron, laid an experimental pavement n which synthetic rubber GR–S Type V was sl (*10*). The asphaltic concrete mixture, o orming to requirements of Ohio specifica-ic T–35, was typical of those in general s in that area except for the addition of u ber to the binder. The asphalt cement of 035 penetration was heated to 300° F. and y hetic rubber was added and mixed for 2 o s to produce blends containing 5 and 7.5 e ent by weight of rubber. The rubber-s alt blends were then allowed to stand v night before mixing with the aggregate. Cl characteristics of these rubber-asphalt l ds, as reported by the City of Akron (*10*), r given in table 1. These data show that h addition of rubber to the asphalt cement a ed a reduction in penetration and ductility n increased the softening point. Solubility es of the rubber-asphalt blends in carbon dis fide indicated that the rubber was not ol ble. The test results also indicate that the insoluble rubber did not affect the Oliensis spot test.

The asphaltic-concrete mixtures containing the rubber powder were handled in the normal manner during mixing, transportation, laying, and compacting. An inspection of the pavement sections made in February 1949 showed no visual difference between those containing rubber and those containing asphalt without rubber.

Objective of Study

The above review of the literature indicates that rubber in many forms has been added to asphaltic materials for road-building purposes. These include rubber latex combined with asphalt cements and asphalt emulsions, plasticized rubber with fluid and semisolid asphaltic materials, and various kinds of powdered rubber combined with cutback asphalts and road tars for surface treatments and cold-laid mixtures and with asphalt cement for hot-laid mixtures. However, the chief objective of the investigation reported here was to determine the effect of various kinds of rubber powders on the properties of the petroleum asphalts normally used in the construction of high-type paving surfaces.

In the preparation of asphaltic-concrete mixtures, the rubber powder usually has been added to the hot aggregate in the mixing box at the paving plant prior to the addition of the asphalt. This is the most convenient method and is recommended by the Natural Rubber Bureau. However, the rubber powder may not be completely blended with the asphalt during the mixing and laying of the paving mixture and the degree of alteration which has taken place cannot be determined. As will be shown later, on account of the high degree of insolubility of the rubber powders in the solvents normally used in the extraction of bitumens from paving mixtures, the material extracted and recovered from rubberized asphalt mixtures may differ greatly from the binder as it exists in the pavement.

Therefore, this report deals primarily with the effect of the various kinds of powdered rubber on the physical properties of selected asphalt in order to obtain data that may prove of value in establishing the relative efficiency of the different rubbers as admixtures to the asphalt binders.

Summary of Observations

The report shows that a number of variables affect the characteristics of blends of rubber with petroleum paving asphalts. In some cases the effect of these variables is not consistent. Disregarding the inconsistencies, the following statements may be made:

The type of rubber used in the blend affects the characteristics of the asphalt in varying degree: Natural and GR–S Type II synthetic rubbers produce large changes; polybutadiene rubber produces medium changes; and reclaimed, processed, tire-scrap, and GR–S Type V synthetic rubbers produce only small changes.

The changes in the properties of a given asphalt become more pronounced with increases in the rubber content of the blend.

With a given type and amount of rubber in the blend, the changes in the properties of the asphalt vary with its character and source.

The temperature, time of heating, and amount of stirring in the preparation of the blend all have their effects on the character of the blend.

Insofar as individual test characteristics are concerned, the addition of rubber to asphalt has the following effects:

For all asphalts studied, the softening point and viscosity were increased and the susceptibility to temperature change was decreased. The flow was also decreased except for the blend of Venezuelan asphalt with 5 percent of reclaimed rubber.

In general, the penetration of the asphalts was decreased by the addition of rubber. Exceptions to this were the blends of California asphalt with natural rubber and with 5 percent of synthetic rubber.

Tests with one asphalt showed that the elasticity was increased in varying degree by the addition of various rubbers.

The ductilities of the asphalts at 77° F. were greatly decreased by the addition of the various rubbers. Exceptions to this were the blends of California asphalt with natural rubber and GR–S Type II synthetic rubber.

Previous studies have shown that all paving asphalts have a much lower ductility at 39.2° F. than at 77° F. when tested at a rate of 5 centimeters per minute. Blends of the three asphalts with 7.5 percent of various rubbers also had much lower ductilities at 39.2° F. than at 77° F. except the blends containing GR–S Type II rubber. With the exception of these and the blend of California asphalt and natural rubber, the low-temperature ductility of the asphalts was not influenced greatly by the addition of rubber.

The results of thin-film oven tests show that there is a wide difference in the resistance of different rubber-asphalt blends to the high temperatures encountered in the processing of bituminous paving mixtures.

All of these indications provide useful information. However, the properties of rubber asphalt blends as they exist in rubber-asphalt pavements cannot be determined by available methods of extraction and recovery. There-

Table 1.—Analysis of asphalt and rubber-asphalt blends used as binders in asphaltic concrete on Exchange Street, Akron, Ohio, 1948

	Unblended asphalt	Asphalt with 5% rubber	Asphalt with 7.5% rubber
Foaming when heated to 350.6° F	None	None	None
Flash point, °F	617	599–608	599–608
Penetration at 100 g., 5 sec., 77° F	75	63	57
Softening point, R and B, °F	120–127	124–129	126–134
Ductility at 77° F., cm	100+	12–28	14.5–20
Loss at 325° F., 5 hrs., percent	0.1	0.1	0.1
Residue, percent of original penetration	68.0–85.6	66.8–82.5	81.0–87.7
Material soluble in CS_2, percent	99.5	94.9	92.4
Insoluble matter in CCl_4, do	0.3	5.2	7.6
Oliensis spot test	Neg.	Neg.	Neg.

Table 2.—Types of rubber powders used in investigation

[illegible]	Producer	Type, source, and method of preparation of rubber powder
R-1	A	Natural: East Indies.
R-2	B	Reclaimed, 16 mesh.
R-3	B	Reclaimed, 12 mesh.
R-4	B	Reclaimed, 8 mesh.
R-5	C	Processed.
R-6	D	GR-S Type V, 70/30 butadiene-styrene; prepared by vulcanization, coagulation, and pulverization in presence of detackifier.
R-7	D	GR-S Type II, 75/25 butadiene-styrene; prepared by coagulation in presence of resin polymer detackifier.
R-8	D	GR-S Type II, 75/25 butadiene-styrene; prepared by coagulation in presence of soap detackifier.
R-9	D	GR-S Type V, 70/30 butadiene-styrene; prepared by vulcanization, coagulation, and pulverization in presence of a detackifier.
R-10	D	Polybutadiene; made from polybutadiene latex by coagulation in presence of a resin polymer as detackifier.
R-13	D	GR-S Type II, 75/25 butadiene styrene; prepared from GR-S Type II latex, spray dried with polyvinal chloride as detackifier, and ground with perlite to pulverize.
R-14	D	Polybutadiene; prepared from polybutadiene latex, spray dried with polyvinal chloride as detackifier, and ground with perlite to pulverize.
R-15	E	Reclaimed; powdered.
R-16	E	Ground vulcanized tire treads.

[1] Designated by producers as Powderpol and considered by producer to be quite similar to R-6.

fore, since experimental asphalt pavements containing various rubber powders have not as yet shown significant differences in behavior between the sections containing asphalt alone and those containing rubber-asphalt blends, it is not possible at this time to evaluate the influence of the rubber by means of tests of the blends made in the laboratory.

Rubber Powders Investigated

Representative rubber powders obtained either from construction projects or from interested producers were included in the investigation. The sample identification and a brief description of all the rubbers used are given in table 2. Material R–1 from producer *A* was the natural, partially vulcanized, rubber powder known as Mealorub, which has been previously described. Materials R–2, R–3, and R–4, differing only in particle size, were reclaimed rubbers from producer *B*. Material R–15 from producer *E* was also a reclaimed rubber. Reclaimed rubber is described (*11*) as follows:

> During the reclaiming operations the vulcanized rubber compound in the original scrap becomes devulcanized, i. e., it again becomes processable, so that it can again be compounded and vulcanized. Reclaim is valued largely for its rubber hydrogen content, other ingredients consisting essentially of fillers and softeners, some of which were in the original rubber scrap and some of which were added during reclaim manufacturing process. Reclaim may, therefore, be considered as a partially compounded rubber.

Material R–5, furnished by producer *C*, was designated as processed rubber. This producer stated that processed rubber is a compounded rubber with a specific gravity of 1.17 ±0.03, ground to pass a 20-mesh sieve.

The seven synthetic rubbers from producer *D* consisted of two GR–S Type V powders, R–6 and R–9, similar to the material used in the Exchange Street pavement, Akron, Ohio. Rubbers R–7, R–8, and R–13 were of GR–S Type II, and R–10 and R–14 were polybutadiene powders prepared from the polymerization of polybutadiene latex.

The results of a laboratory study of the effect on petroleum asphalts of butadiene-based synthetic rubber powders, including the GR–S Type V rubber used in the Exchange Street experimental sections in Akron, were given in a report published in 1951 (*12*). This report showed that synthetic rubbers added to hot asphalts raised the softening point and reduced the cold flow, the susceptibility to temperature, and the penetration of the asphalt.

Rubber R–16 from producer *E* was ground, vulcanized tire-tread scrap. The ground tire-tread scrap is similar to that used by A. E. H. Dussek in his composite surfacing material, patented in Great Britain in 1935 and in the United States in 1938 (*13, 14*). Dussek, in 1951, discussed the performance of a paving mixture containing ground vulcanized tire-tread scrap laid in 1937 at Clifton Rise, New Cross, England (*15*). The quantity of rubber used in this paving mixture was, however, much greater than the amounts of other rubber powders that have been used in rubberized asphaltic paving mixtures. At the time of his report, the pavement was said to be in excellent condition.

With the exception of rubbers R–7, R–8, R–10, R–13, and R–14, all the rubbers listed in table 2 have been used in experimental sections of rubberized asphalt paving mixtures. Although not used in this study, another rubber product has been developed recently and has been used in asphaltic paving mixtures (*16, 17*). This is a material in which a GR–S Type II rubber latex, simila to that used in rubber R–13 of this report is co-precipitated with a mineral filler. Th material is said to mix more readily wit asphaltic materials either alone or in asphalti aggregate mixtures.

In this study, no attempt was made t examine these various rubbers chemically Determinations of specific gravity, bulk den sity, ash content, grading, and solubility i organic solvents were made. The results o these and other tests are given in table 3.

The wide range in specific gravity of th various rubbers accounts, no doubt, for th tendency of some of them to settle out or t float to the surface when mixed with asphalti materials and allowed to stand for long per ods of time in a fluid condition. This tenc ency was noticed particularly for thos rubbers that were not dispersed readily in th asphaltic materials. Also, because of thi wide difference in specific gravities, propor tioning the rubber powders on a weight basi as used in this study, may affect the compa ative results slightly.

Physical Properties Differ

The solubility of the rubbers in the organ solvents not only aids in distinguishing th different types of rubber but also gives a indication of the probable solubility of th rubber portion of a rubber-asphalt blen when it is extracted from an asphalt mixtu for the determination of bitumen conten The results of the solubility tests showed th the two GR–S Type V synthetic rubbers R- and R–9, and the partially vulcanized natur rubber R–1, had similar solubilities in th three solvents. Of the reclaimed rubbers, R– had a much greater solubility in all solven than R–2, R–3, and R–4. It is of intere to note that the processed rubber R–5 a the ground, vulcanized tire-tread scrap R– had similar solubility in all solvents. The was considerable variation in the solubility the GR–S Type II rubbers R–7, R–8, a

Table 3.—Results of tests on rubber powders

	Type and identification of rubber powders													
	Natural (R-1)	Reclaimed, 16 mesh (R-2)	Reclaimed, 12 mesh (R-3)	Reclaimed, 8 mesh (R-4)	Processed (R-5)	GR-S Type V (R-6)	GR-S Type II (R-7)	GR-S Type II (R-8)	GR-S Type V (R-9)	Polybutadiene (R-10)	GR-S Type II (R-13)	Polybutadiene (R-14)	Reclaimed powder (R-15)	Ground scrap (R-
Specific gravity, 77°/77° F.	0.980	1.122	1.118	1.121	1.170	1.002	0.972	0.924	0.975	0.935	1.070	1.017	1.189	1.12
Bulk density in air g./cm.³	0.37	0.37	0.31	0.33	0.37	0.33	0.28	0.21	0.31	0.19	0.30	0.39	0.25	0.3
Organic matter insoluble—														
In 86° B naphtha percent	93.8	71.1	78.1	72.1	85.8	93.9	79.3	71.8	92.4	64.0	66.0	47.2	61.6	86.
In cold CS_2 do	93.8	68.3	71.5	70.7	84.0	93.8	44.1	54.1	93.5	72.3	43.7	46.7	60.4	85.
In hot CS_2 do	91.4	64.3	67.5	67.2	82.0	91.9	39.7	41.3	90.6	65.6	41.0	38.4	58.3	82.
In hot C_6H_6 do	91.0	64.5	67.6	66.7	82.0	91.1	60.0	77.3	92.3	84.8	61.0	64.5	57.4	79.
Ash by ignition do	6.2	9.4	8.1	8.7	6.6	2.6	1.5	1.9	2.6	1.4	14.3	7.5	14.0	7.
Loss on heating—														
At 212° F., 24 hr. percent	+0.65	0.35	0.48	0.38	0.65	0.30	0.31	0.32	0.17	0.41	0.30	0.27	2.69	+1.7
At 325° F., 2 hr. do	+0.86	0.51	0.51	0.19	+0.16	0.63	0.61	0.35	0.34	1.06	+1.61	+0.11	1.52	+1.(
At 325° F., 24 hr. do	+0.94	1.60	1.25	1.30	+1.12	+1.64	0.20	+5.37	0.39	+4.83	+1.40	+0.70	1.99	+1.3
Grading, percentage passing—														
No. 10 sieve	100	99	99	96	100	100	78	96	100	100	100	100	66	1(
No. 20 sieve	97	69	18	21	100	100	46	47	100	83	100	88	22	9
No. 30 sieve	83	33	7	9	88	98	30	27	96	73	89	75	11	(
No. 40 sieve	57	14	2	3	50	90	18	15	66	59	57	49	4	3
No. 50 sieve	31	4	1	1	26	83	10	8	38	36	31	27	2	
No. 80 sieve	8	1	------	------	10	71	4	3	13	15	14	7	1	
No. 100 sieve	3	----	------	------	5	61	2	------	9	7	10	4	------	----

Table 4.—Analysis of original asphalts used in rubber-asphalt blends

	Asphalt AC–1	Asphalt AC–2	Asphalt AC–3
Specific gravity, 77°/77° F.	1.020	1.012	1.013
Penetration, 100 g., 5 sec.—			
At 50° F.	20	24	16
At 77° F.	89	94	90
At 95° F.	232	238	270
Softening point, °F.	118.0	120.0	113.2
Ductility, 77° F., cm.	+250	195	200
Standard oven test, 5 hr., 325° F.:			
Loss, percent	0.14	0.02	0.09
Tests on residue:			
Penetration, 77° F.	77	76	78
Softening point °F.	122.0	123.8	114.4
Ductility, 77° F., cm.	+250	160	+250
Thin-film oven test, 5 hr., 325° F.:			
Loss, percent	0.28	+0.01	0.33
Tests on residue:			
Penetration, 77° F.	51	56	51
Softening point °F.	131	134	121
Ductility, 77° F., cm.	237	52	215
Bitumen soluble in CS_2, percent	99.89	99.94	99.95
Organic insoluble, do	0.09	0.02	0.00
Inorganic insoluble, do	0.02	0.04	0.05
Oliensis spot test, standard naphtha	Neg.	Neg.	Neg.

t–13, except in hot carbon disulfide. The wo polybutadiene rubbers R–10 and R–14 iffered in solubility in all the solvents.

There was a wide range in the percentage f ash by ignition, ranging from 1.4 percent or rubber R–10 to 14.3 percent for rubber t–13. However, only in the case of rubbers t–13 and R–14, in which perlite was used to revent coagulation of the rubber particles, as the percentage of ash considered in the roportioning of the rubber-asphalt blends. The producer suggested that 6 percent of hese materials be used for comparison with percent of the other synthetic rubber powders.

The reclaimed rubbers R–2, R–3, R–4, and t–15 and synthetic rubbers R–7 and R–9 all ost weight when heated under all the condi- ions described in the table. Rubbers R–1 nd R–16 increased in weight for all conditions f heating. When heated for 24 hours at 25° F., rubbers R–1, R–5, R–6, R–8, R–10, R–13, R–14, and R–16 increased in weight from 0.70 to 5.37 percent, rubber R–8 showed an increase of 5.37 percent, and rubber R–10 an increase of 4.83 percent.

The increase in weight of some of the rubbers indicates that oxidation might occur when they are exposed to high temperatures such as those encountered during the dry mixing with heated mineral aggregates in a mixing plant. The fineness of the rubber powder and the large surface area exposed may be inducive to alterations of the rubber if the temperature is too high or the mixing period too long.

The results of the sieve analyses of the various rubbers showed that there was a wide range in their fineness. Rubbers R–1, R–5, R–9, R–10, R–13, and R–14 had approximately the same fineness, while rubbers R–2, R–3, R–4, R–7, R–8, R–15, and R–16 were coarser, and rubber R–6 was much finer than those in the first named group.

Preliminary Study of Blends

Although other penetration grades of petroleum asphalts were used in the investigation of the rubber-asphalt blends, the test results of the asphalts most extensively used are given in table 4. These were of the 85–100 penetration grade. AC–1 is a vacuum-refined Venezuelan asphalt that is refined along the eastern seaboard and is used extensively in that area. AC–2 came from a midwest producer and, although the source is not known, it was furnished by the producer who supplied the asphalt used in the rubberized asphaltic concrete constructed on Exchange Street, Akron, Ohio, in 1948. AC–3 is a California asphalt of the low-gravity, low-sulfur, and low-asphaltene type that is extensively used on the west coast.

All of the rubbers described in tables 2 and 3 were first blended with the Venezuelan asphalt AC–1. Five percent of rubber by weight of the blended material was used except for rubbers R–13 and R–14, for which 6 percent was used. (Since this percentage is considered equivalent to 5 percent of the other rubbers, it is reported as 5 percent in table 5.) These blends were prepared in an oil-jacketed heating kettle equipped with an electric stirrer for circulating the oil, a thermostatic temperature regulator, and a motor-driven stirrer for mixing the rubber and asphalt (fig. 1).

With this equipment the temperature could be controlled within a range of ±5° F. of the desired mixing temperature. For the blends prepared at 300° F. it was necessary to maintain the temperature of the oil bath at approximately 315° F. For blends prepared at higher temperatures, this differential in temperature was a few degrees more.

In preparing the rubber-asphalt blends, 300 grams of asphalt were heated on a hotplate to the desired mixing temperature and weighed into the preheated mixing kettle. The desired amount of rubber powder was then added to the asphalt with the stirrer operating at a speed of approximately 200 r.p.m. By means of a rheostat connected to the motor, this approximate speed was maintained throughout the heating and mixing period. A thermometer placed in the mixture indicated the temperature of the material throughout the mixing process.

In the preliminary study, the original asphalt and all of the rubber-asphalt blends were heated for 2 hours at 300° F. The literature indicated that a wide variety of temperatures had been used in previous investigations, but a temperature of 300° F. was selected as being reasonably close to the temperatures normally used in hot paving plants for the 85–100 penetration grade of asphalt. Preliminary work also indicated that usually the maximum effect of the rubbers can be obtained at this temperature within a 2-hour heating period. Endres, et al. (*12*), used a temperature of 302° F. and a heating period of 5 hours with only occasional stirring, while in this study the stirring was continuous.

After completion of the 2-hour blending period, the rubber-asphalt blends and the unblended asphalts were tested by the AASHO

Table 5.—Results of tests on unblended asphalt AC–1 and blends containing 5 percent of various rubbers after mixing for 2 hours at 300° F.

	Unblended asphalt AC–1	Rubbers blended (5 percent) with asphalt AC–1												
		R–1	R–2	R–3	R–4	R–5	R–6	R–7	R–8	R–9	R–10	R–13	R–14	R–15	R–16
Specific gravity, 77°/77° F.	1.019	1.015	1.022	1.022	1.027	1.028	1.015	1.016	1.014	1.015	1.007	1.020	1.020	1.028	1.025
Penetration, 100 g., 5 sec.—															
At 50° F.	22	22	21	18	19	20	20	19	23	18	23	22	27	18	19
At 77° F.	87	66	76	80	78	73	71	68	73	67	71	73	73	73	67
At 95° F.	213	139	189	188	186	172	156	157	157	156	147	157	149	179	156
Softening point, °F.	118	143	123	122	119	124	125	128	135	126	136	134	136	124	129
Ductility 77° F., cm.	195	28	31	38	31	13.5	23	32	40	19.5	22.5	66	38	28	20
Rebound, percent	-----	63	48	61	58	26	13	36	55	6	43	66	60	46	49
Ductility, 60° F., cm.	+250	17.5	12	11	10	11.5	12.5	62	74	8	18.5	41	15	12	13
Rebound, percent	-----	40	38	39	38	50	22	45	57	38	45	54	52	47	58
Flow at 140° F., 1 hr., cm.	12.3	3.7	11.5	10.8	11.3	10.5	10.0	7.9	6.6	9.7	6.0	3.9	3.7	12.6	11.2
Thin-film oven test:															
Loss 325° F., 5 hr., percent	0.25	0.19	0.35	0.35	0.40	0.27	0.24	0.24	0.28	0.21	0.26	0.30	0.30	0.31	0.23
Tests on residue:															
Penetration, 100 g., 5 sec.—															
At 50° F.	13	17	13	13	13	13	16	17	18	15	19	-----	--	15	15
At 77° F.	51	65	47	46	46	46	52	52	56	48	57	57	62	47	47
At 95° F.	114	242	96	100	97	98	109	111	116	99	113	--- --	---	98	100
Softening point, °F.	131	129	138	136	136	138	137	137	142	142	142	151	160	142	140
Ductility, 77° F., cm.	+220	+250	8	10	7.5	12.5	12	50	40	10	25	48	14	14	12
Rebound, percent	-----	--- -	34	35	47	52	19	60	58	45	48	59	55	64	58
Soluble in CS_2, do	99.89	97.59	95.90	94.60	95.42	95.30	95.22	97.94	98.37	94.90	97.15	97.69	96.13	95.57	95.09
Organic insoluble, do	0.09	2.12	3.63	4.97	4.12	4.24	4.70	1.96	1.49	4.93	2.80	1.62	2.53	3.22	4.32
Inorganic insoluble, do	0.02	0.29	0.47	0.43	0.46	0.46	0.08	0.10	0.14	0.17	0.05	0.69	1.34	1.21	0.59
Organic insoluble in 86° naphtha, do	19.59	21.79	21.77	24.21	21.37	24.30	24.19	21.67	21.56	24.39	23.36	20.09	21.45	21.99	22.83
Oliensis spot test, standard naphtha	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(2)	(3)	(1)	(3)	(2)	(2)	(1)	( )

[1] Negative. [2] Positive. [3] Slightly positive.

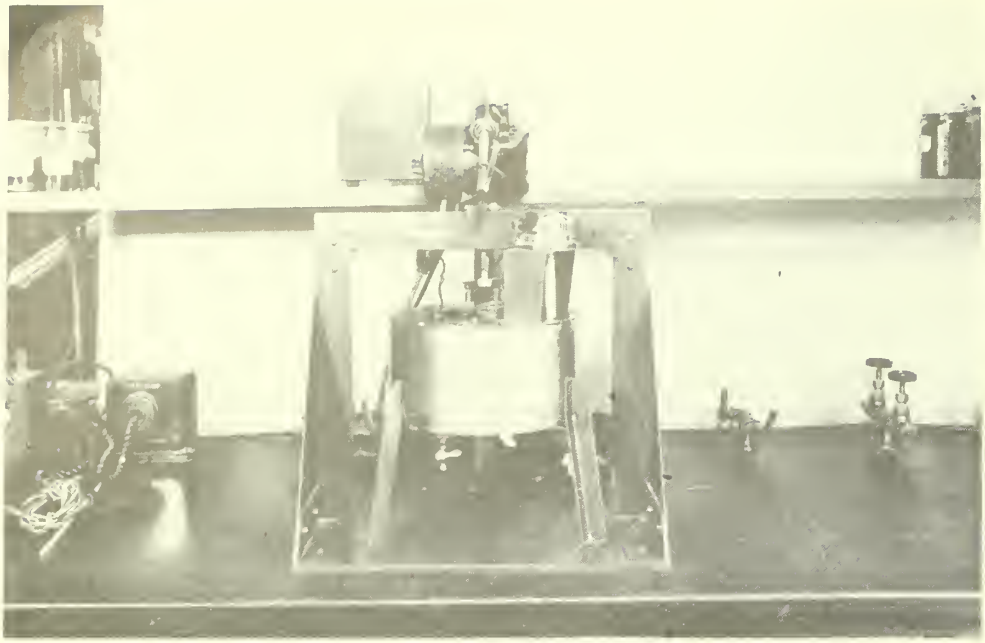

Figure 1.—Mixing kettle used for preparing rubber-asphalt blends.

test methods usually employed for the control of asphalt cements and also by methods that, although not standard for these materials, might furnish some information on the effect of the rubber on the properties of the asphalts.

The results of tests on the 5-percent rubber blends with asphalt AC–1 are given in table 5. An examination of the test data in this table shows that, as compared with the unblended asphalt, the addition of 5 percent of the rubber powders reduced the penetration at 77° F. from 7 points for the reclaimed rubber R–3 to 21 points for the natural rubber R–1. The softening point was raised 25° F. for rubber R–1 and only 1° F. for rubber R–4. As compared with the unblended asphalt, the reduction in ductility at 77° F. for all blends was exceedingly high—the most ductile blend, that containing rubber R–13, having only 34 percent of the ductility of the unblended asphalt.

In the thin-film oven tests, the rubber-asphalt blends behaved like the unblended asphalt except that the ductility at 77° F. of the blend containing rubber R–1 increased enormously as compared with the original ductility of the blend. Blends with rubbers R–7 and R–10 also showed some increase. In the thin-film oven test all the blends and the unblended AC–1 asphalt showed a material reduction in penetration at 77° F., except the blend containing rubber R–1, which had approximately the same consistency, before and after the test. The softening points of residues from thin-film oven tests increased from 7° to 24° F., except for the softening point of the blend containing rubber R–1, which decreased 14° F.

The organic matter of the blends insoluble in carbon disulfide varied from 4.97 for the blend with rubber R–3 to 1.49 for that with R–8. The blends containing the GR–S Type V rubbers R–6, and R–9, the reclaimed rubbers R–2, R–3, and R–4, the processed rubber R–5, and the ground tire scrap R–16, all were highly insoluble in carbon disulfide.

In making the Oliensis spot test, results of which are given in table 5, standard naphtha was used as solvent by the procedure given in AASHO method T 102–42. It is apparent that the character of the spot is influenced by the type of rubber. Except for the blend with ground scrap rubber R–16, the on blends that showed positive spots were tho containing synthetic rubbers. This conc tion may be due either to the solubility the rubber in the asphalt and solvent, or the presence of large amounts of the organ or inorganic material added to the rubber a detackifier. Blends containing rubbe R–13 and R–14 contained the largest amou of ash and gave the darkest spot.

Rubbers Selected for Detailed Stud

On the basis of the preliminary study, fi ubbers were selected for a more detail study so that the various kinds of rubb would be represented and also so that rubbe would be included that had the minimum an maximum effect on the asphalt. Tho selected were the natural rubber R–1, t reclaimed rubber R–2, the GR–S Type rubber R–6, the polybutadiene rubber R–1 and the GR–S Type II rubber R–13. In few of the tests other rubbers were als included.

Blends of the five rubbers with the thr asphalts were prepared to determine the effe of rubber content and of the time and ten perature of mixing on the test characteristic In order to determine the effect of differe amounts of rubber on the properties of th blends, 5.0, 7.5, and 10.0 percent of eac rubber were added to the asphalts. As mer tioned previously, due to the large amount inorganic matter used as a detackifier in th manufacture of rubber R–13, the produce held that 6 percent of this material would b equivalent to 5 percent of the other syntheti rubbers. Therefore, for this rubber th blends contained 6.0, 9.0, and 12.0 percent the powder as received but in the tables an charts these percentages are reported as 5. 7.5, and 10.0. As in the preliminary study the blends were prepared by mixing th asphalt and rubber at 300° F. for 2 hours.

In the preliminary study considerabl change, as evidenced by a noticeable increas in consistency, occurred in some of the blend during the 2-hour mixing period. In order t determine the rate and extent of these altera tions, sufficient material was removed inte mittently during the 2-hour period of mixin for softening-point determinations. This wa

Table 6.—Effect of time of mixing at 300° F. on softening point of asphalts and rubber-asphalt blends

	Softening point (in °F.) of blend containing rubber in percentages indicated														
	Unblended asphalt	R–1			R–2			R–6			R–10			R–13		
		5%	7.5%	10%	5%	7.5%	10%	5%	7.5%	10%	5%	7.5%	10%	5%	7.5%	10%
Asphalt AC–1 and blends after mixing																
5 min	118.0	125.0	131.0		117.0	122.4	123.3	125.2	127.8	131.8	134.5	144.8	153.8	127.2	134.5	144.5
30 min	118.0	133.5	143.3	152.0	122.7	125.8	127.5	124.9	129.0	134.5	136.5	145.8	158.0	132.0	167.0	188.0
60 min	118.0	140.0	153.0	166.5	124.5	126.0	129.8	127.0	130.3	132.3	135.5	146.3	159.3	134.0	172.5	188.0
90 min	118.0	143.0	158.6	172.5	121.3	126.5	130.8	125.6	130.5	133.5	136.0	147.3	161.1	134.3	175.5	184.5
120 min	118.0	142.5	157.3	172.0	122.9	128.8	130.8	125.0	130.3	135.3	136.0	146.5	160.6	134.0	174.0	181.0
Asphalt AC–2 and blends after mixing																
5 min	120.0	125.5	129.7	133.3	121.0	123.0	124.0	125.3	128.8	134.0	138.6	150.5	160.3	127.3	133.3	141.5
30 min	121.0	134.6	143.0	151.5	123.5	125.8	129.3	126.0	130.0	135.0	140.3	153.5	166.3	130.5	156.0	178.0
60 min	121.2	142.0	156.5	168.8	123.3	128.0	130.5	127.8	131.2	135.0	140.3	153.0	167.0	137.4	163.5	189.3
90 min	121.5	147.5	164.5	181.5	123.5	128.0	131.3	127.8	131.4	135.0	140.5	153.5	166.8	143.0	167.5	186.5
120 min	121.5	147.3	162.3	180.0	125.5	127.5	131.8	127.0	131.6	135.0	141.0	153.5	168.5	142.8	158.0	178.0
Asphalt AC–3 and blends after mixing																
5 min	113.2	121.0	126.0	126.5	113.8	116.0	115.4	119.0	121.5	125.5	124.5	130.5	136.3	119.5	123.3	131.0
30 min	113.1	127.0	137.0	147.0	115.0	117.2	118.5	120.8	122.0	126.5	124.0	129.8	135.0	125.8	138.0	156.0
60 min	113.5	123.5	130.0	136.3	116.0	117.0	118.6	120.0	122.5	126.0	124.0	129.9	136.0	128.0	140.0	156.2
90 min	114.5	120.0	123.0	131.0	116.3	117.8	119.8	120.0	123.0	126.5	124.0	130.0	136.3	128.8	141.5	156.4
120 min	114.8	118.0	121.8	129.3	116.3	118.3	120.3	120.0	123.4	126.8	124.0	129.8	137.3	129.3	141.3	155.5

Table 7.—Results of tests on asphalts and rubber-asphalt blends after mixing for 2 hours at 300° F.

Blend identification	Penetration, 100 gm., 5 sec.			Softening point	Ductility, 77° F.		Flow, 140° F., 1 hour	Thin-film oven test, ⅛-in. film, 5 hr., 325° F.					Solubility in CS_2
								Loss on heating	Tests on residue				
									Penetration, 77° F.	Softening point	Ductility, 77° F.		
	At 50° F.	At 77° F.	At 95° F.		Elongation	Rebound					Elongation	Rebound	
				° F.	Cm.	Percent	Cm.	Percent		° F.	Cm.	Percent	Percent
Unblended AC-1	22	87	213	118.0	195		12.3	0.25	51	131	+220		99.89
Blend of AC-1 and—													
R-1, 5 percent	22	66	139	142.5	28	63	3.7	.19	65	129	+250		97.59
R-1, 7.5 percent	18	57	115	157.3	20	60	1.6	.24	62	134	135		96.30
R-1, 10 percent	19	51	99	172.0	8	63	1.1		67	142	65	35	96.53
R-2, 5 percent	21	76	189	122.9	31	48	11.5	.35	47	138	8	34	95.90
R-2, 7.5 percent	17	68	156	128.8	16	37	7.0	.41	46	142	9	49	94.08
R-2, 10 percent	19	64	143	130.8	10	37	4.5	.56	40	149	8	58	92.26
R-6, 5 percent	20	71	156	125.0	23	13	10.0	.24	52	137	12	19	95.22
R-6, 7.5 percent	19	64	150	130.3	19	19	7.3	.26	46	140	10.5	43	92.79
R-6, 10 percent	17	58	128	135.3	16	23	6.5	.26	46	145	9		91.53
R-10, 5 percent	23	71	147	136.0	22.5	43	6.0	.26	57	142	25	48	97.15
R-10, 7.5 percent	23	65	118	146.5	18	47	1.9	.32	56	159	16.5	50	95.64
R-10, 10 percent	29	65	107	160.6	11.5	52	1.0	.33	56	174	9	55	94.20
R-13, 5 percent	22	73	157	134.0	66	66	3.9	.30	57	151	48	59	97.69
R-13, 7.5 percent	24	66	120	174.0	34	72	1.2	.33	53	171	27	66	96.13
R-13, 10 percent	24	65	110	181.0	25	73	1.0	.33	54	184	22	67	95.11
Unblended AC-2	23	87	224	121.5	159		14.5	+.05	55	134	45		99.94
Blend of AC-2 and—													
R-1, 5 percent	20	67	125	147.3	15	60	3.5	.00	72	131	53		97.69
R-1, 7.5 percent	23	60	115	162.3	8	56	1.2	.00	71	137	22	31	97.16
R-1, 10 percent	19	52	97	180.0	4.5	67	0.8	.04	81	144	18.5	34	95.24
R-2, 5 percent	24	80	188	125.5	15.5	32	9.8	.11	52	141	15.5	36	96.12
R-2, 7.5 percent	24	75	168	127.5	8.5	29	8.4	.08	49	145	8.5	50	93.93
R-2, 10 percent	23	69	148	131.8	9	35	5.2	.22	49	149	9.0	54	92.50
R-6, 5 percent	23	74	171	127.0	17.5	17	8.7	.04	59	136	7.5	37	95.01
R-6, 7.5 percent	22	67	152	131.6	14.5	19	7.8	.04	58	140	6	44	92.64
R-6, 10 percent	21	62	132	135.0	12	20	5.5	.07	54	146	6.5	46	90.05
R-10, 5 percent	30	75	140	141.0	13	46	3.6	.05	63	147	15.5	48	96.85
R-10, 7.5 percent	32	70	120	153.5	7.8	39	1.9	.10	61	162	10	50	95.61
R-10, 10 percent	34	69	108	168.5	4	56	.8	.11	63	179	5	50	94.31
R-13, 5 percent	27	80	158	142.8	43	70	3.2	.06	65	150	31	63	98.02
R-13, 7.5 percent	28	73	130	158.0	32	67	1.9	.16	62	178	24	63	96.85
R-13, 10 percent	28	67	109	178.0	18	63	1.0	.16	60	184	28.5	58	95.64
Unblended AC-3	15	86	264	114.8	+250		15.0	.33	48	122	235		99.95
Blend of AC-3 and—													
R-1, 5 percent	13	95	272	118.0	+250		12.5	.41	65	120	+250		98.98
R-1, 7.5 percent	20	101	267	121.8	+250		10.8	.36	73	120	+250		98.85
R-1, 10 percent	33	123	268	129.3	+250		7.5	.14	82	120	+250		98.40
R-2, 5 percent	15	82	237	116.3	32	57	13.4	.43	48	126	14.5	43	96.06
R-2, 7.5 percent	16	81	223	118.3	19	49	10.2	.50	47	130	17.0	48	93.59
R-2, 10 percent	16	79	198	120.3	16	49	8.5	.57	49	134	15.0	50	91.36
R-6, 5 percent	12	72	200	120.0	25	16	12.7	.37	49	129	27	37	94.95
R-6, 7.5 percent	13	66	186	123.4	21	17	11.6	.34	48	132	24	21	92.69
R-6, 10 percent	11	63	165	126.8	18.5	15	9.3	.32	45	135	24	38	90.40
R-10, 5 percent	16	77	202	124.0	19	43	10.4	.32	62	130	113	65	97.14
R-10, 7.5 percent	17	76	174	129.8	36	50	8.1	.36	62	138	80	73	94.56
R-10, 10 percent	20	76	161	137.3	34	57	3.8	.38	59	161	75	71	92.88
R-13, 5 percent	15	86	217	129.3	+250		6.1	.41	54	130	+250		96.96
R-13, 7.5 percent	16	82	203	141.3	+250		2.9	.41	59	149	190	77	97.02
R-13, 10 percent	19	83	183	155.5	205	75	1.4	.46	58	170	130	73	96.08

done after 5, 30, 60, and 90 minutes of mixing. At the end of 2 hours the blend was removed from the heating kettle and tested for penetration at 50°, 77°, and 95° F., softening point, ductility, oven heat, flow, and solubility tests. The results of the softening-point tests on samples taken at intervals during the 2-hour heating period are given in table 6 and the results of tests on the blends at the end of 2 hours are given in table 7.

Changes occurring in the rubber-asphalt blends during the 2-hour heating and mixing period, as indicated by the softening-point test, are shown in figures 2, 3, and 4 for the blends prepared with asphalts AC–1, AC–2, and AC–3, respectively. The effect of heating on the softening points of the asphalts without rubber also is shown.

It is of interest to note the wide difference in the change of softening point caused by the various kinds of rubber during the mixing period. When the rubber content of the blend was increased there was a much greater rise in softening point for some of the rubbers than for others. This was particularly true for the mixtures containing rubbers R–1 and R–13, with all the asphalts.

The effect of rubber content on the softening point of the blends is more clearly shown in figure 5 where the rubber content is plotted against the actual increase in softening point during the 2-hour mixing period. For some of the rubber-asphalt blends, the maximum increase occurred within the 2-hour period. This was particularly true for the blends of asphalts AC–1 and AC–2 with the synthetic rubber R–13, and of AC–3 with the natural rubber R–1. Therefore, two sets of curves are shown for these particular blends: The solid-line curves show the increase in softening point at the end of the 2-hour mixing and the broken-line curves show the maximum increase that developed during the mixing period.

Considering these maximum increases, the order in which the rubbers increased the softening point was the same for each asphalt used, but the amount of increase varied with the source of asphalt. Thus, not only the type and amount of rubber affects the softening point but the source of the asphalt is also of considerable importance. The influence of the source of the asphalt on the changes in softening point during the time of heating for blends containing 7.5 percent of natural rubber R–1 is shown in figure 6. In general, the effect of source of asphalt is more pronounced for those rubbers that cause the greater changes in softening point. It can be seen in figures 2, 3, and 4 or figure 5 that rubber R–13 as well as Rubber R–1, shown in figure 6, increased the softening point of asphalts AC–1 and AC–2 much more than they did the softening point of AC–3. It also can be seen that rubbers R–2 and R–6 produced little change in softening point of any of the asphalts.

Effect on Penetration

Penetration tests were made at 50°, 77°, and 95° F. on the various rubber-asphalt blends. These results are used to show the change in consistency and to determine a susceptibility factor. The effect of the addition of rubber to the asphalts on penetration is illustrated in figure 7. In general, the addition of rubber to the asphalt resulted in a decrease in penetration at 77° F. For all rubbers the penetration decreased with increase in rubber content, except for the blend of asphalt AC–3 and the natural rubber R–1.

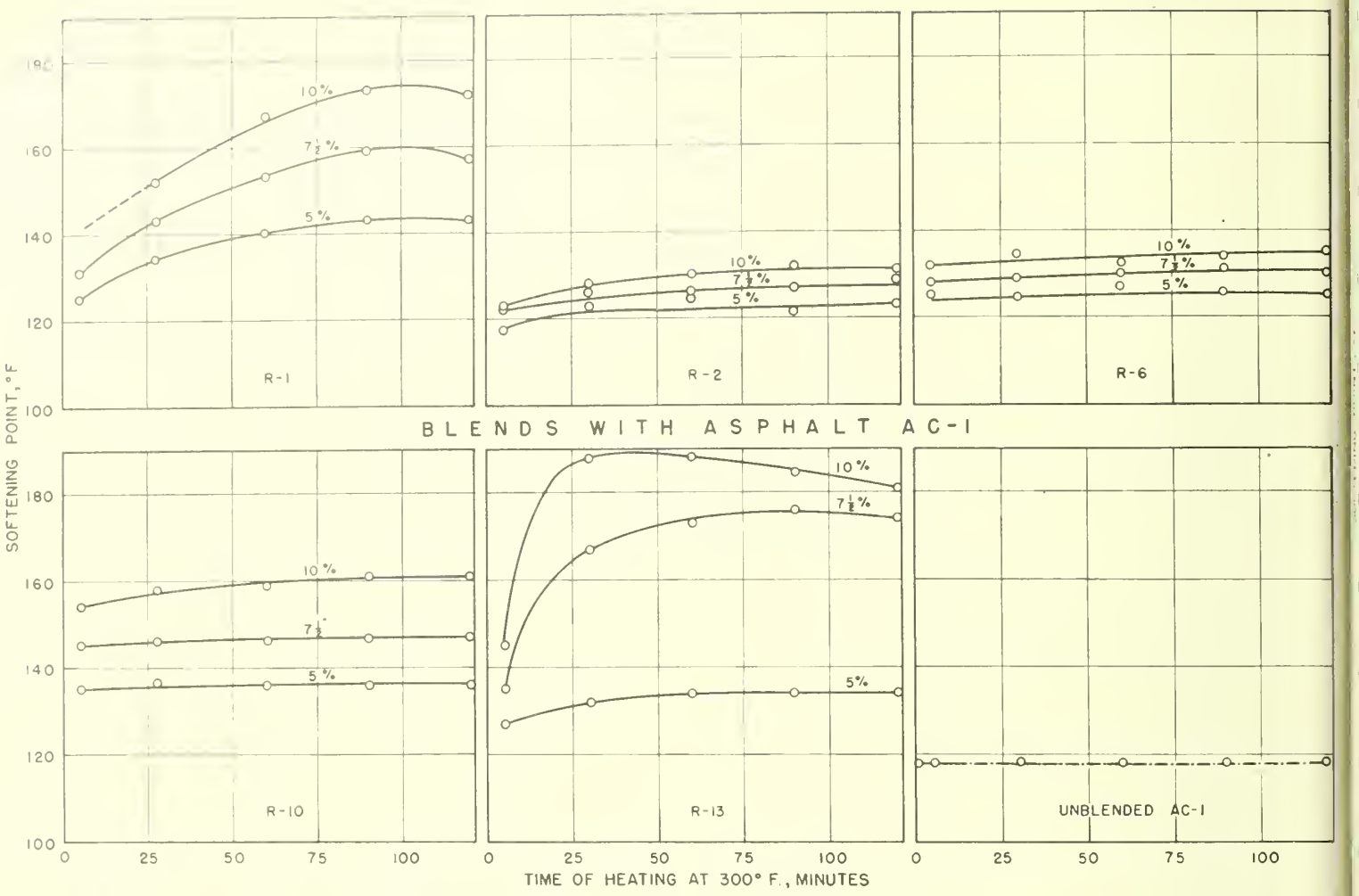

Figure 2.—Change in softening point during heating of blends of asphalt AC–1 with various amounts of powdered rubbers.

As mentioned previously, there was a pronounced decrease in softening point of this same blend during the 2-hour heating period, which indicated some decomposition of the rubber. This change is further reflected in the higher penetration values obtained.

Other investigators have shown that the addition of rubber reduces the susceptibility of asphalt to change in consistency with change in temperature. A previous study of asphalt cements has shown that the slope of log-penetration-temperature curves gives a true measure of the susceptibility of asphalts to temperature (*18*). Therefore, the penetration values obtained at 50°, 77°, and 95° F. as given in table 7 for the various rubber-asphalt blends were plotted, and the slopes of the resulting lines calculated from the following equation:

$$\text{Slope}=\frac{\log P_2-\log P_1}{T_2-T_1}$$

where P_2 and P_1 are penetrations at the two temperatures T_2 and T_1 in degrees Fahrenheit.

For most of the blends the plot of log-penetration versus temperature resulted in straight lines through the three points. Exceptions to this were the blends prepared from asphalt AC–3 and rubbers R–1 and R–13. Here the three points were not linear and for comparative purposes the slopes were calculated from straight lines through the penetrations at 77° and 95° F. The results of the calculated slopes are given in table 8. In general, the susceptibility of the asphalt as measured by the slope of the log-penetration-temperature curves was decreased by the addition of rubber, the amount of decrease being greater with increase in rubber content. The effect of the rubber on lowering the susceptibility varied with the type of rubber and also with the asphalt used. For each of the three asphalts used here, the greatest reduction was produced with rubbers R–10 and R–13 and the least reduction with rubbers R–2 and R–6.

Susceptibility to Temperature Change

The report on paving asphalts of the 50–60 and 85–100 penetration grades (*18*) showed that the slope of the log-penetration-temperature curves for the 50–60 grades varied from 0.0172 to 0.0307, with an average of 0.0230, and for the 85–100 grade varied from 0.0188 to 0.0324, with an average of 0.0242. The slope of the log-penetration-temperature curves of highly blown asphalts (*19*), of approximately 50 penetration, used in sealing expansion joints in concrete pavements, varied from 0.0090 to 0.0125, with an average of 0.0105. Although other investigators have shown that rubber reduces the susceptibility of paving asphalts to temperature change, it is of interest to note that the slope values of many of the rubber-asphalt blends given in table 8 are within the range of the values of the paving asphalts and that only two blends have slope values within the range of those of the highly blown asphalts.

The decrease in susceptibility to change in consistency with change in temperature by

Table 8.—Slope of log-penetration-temperature curves of the original asphalts and the rubber-asphalt blends

Rubber in blend	Slope of curve for blend with asphalt		
	AC–1	AC–2	AC–3
None	0.0218	0.0226	0.0265
R–1, 5 percent	.0179	.0165	[1] .0258
R–1, 7.5 percent	.0174	.0155	[1] .0238
R–1, 10 percent	.0162	.0150	[1] .0189
R–2, 5 percent	.0217	.0207	.0264
R–2, 7.5 percent	.0204	.0197	.0249
R–2, 10 percent	.0194	.0184	.0231
R–6, 5 percent	.0198	.0202	.0250
R–6, 7.5 percent	.0196	.0201	.0253
R–6, 10 percent	.0195	.0185	.0235
R–10, 5 percent	.0176	.0150	.0234
R–10, 7.5 percent	.0146	.0130	.0204
R–10, 10 percent	.0128	.0111	.0185
R–13, 5 percent	.0187	.0165	[1] .0228
R–13, 7.5 percent	.0146	.0140	[1] .0220
R–13, 10 percent	.0128	.0115	[1] .0192

[1] Values taken from curves from 77° to 95° F.

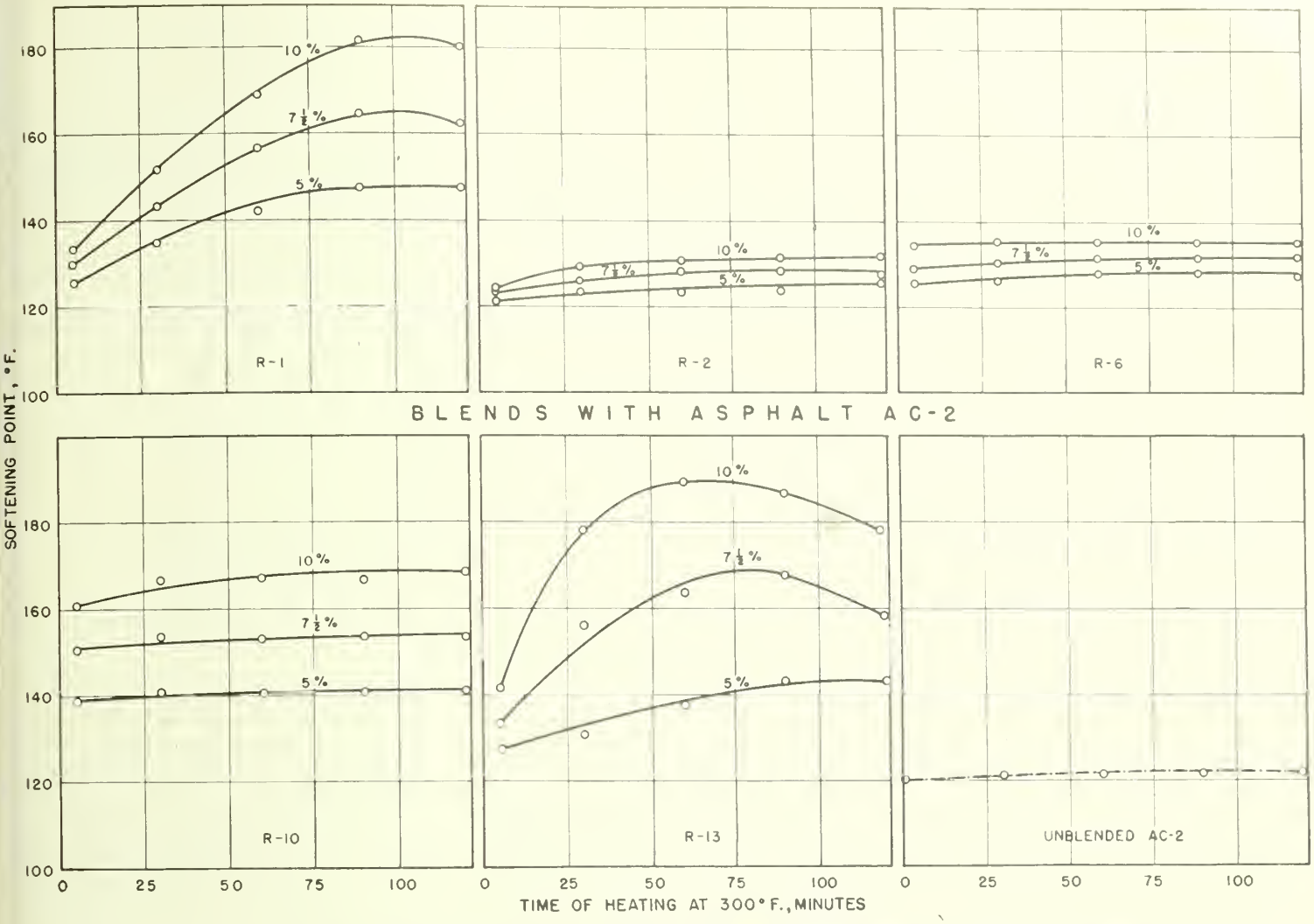

Figure 3.—Change in softening point during heating of blends of asphalt AC–2 with various amounts of powdered rubbers.

the addition of rubber should result in a rubber-asphalt blend that has a higher penetration at low temperature and a lower penetration at high temperature than the untreated asphalt. Such changes in penetration characteristics would make the pavement less brittle in cold weather and more stable in hot weather. In order to determine the effect of the various rubbers on the penetration at low and high temperatures, penetration values at 32° and 100° F. were extrapolated from the log-penetration-temperature curves. Although these are not the extreme temperatures that the asphalt might be subjected to in service they are suitable for comparative purposes. The results of the extrapolated penetration values are given in table 9.

Except for rubbers R–2 and R–6 with asphalt AC–1, the penetrations of the rubber-asphalt blends at 32° F. were higher than for the asphalts themselves. Rubbers R–10 and R–13 with all three asphalts and rubber R–1 with AC–3 showed a marked increase in penetration with increase in rubber content. The penetrations of all the rubber-asphalts were lower at 100° F. than the penetration of the corresponding unblended asphalt. For each rubber and asphalt the penetration was decreased by increase of the rubber content.

It is apparent that, in general, the rubbers do cause an increase in penetration at low temperature and a decrease in penetration at high temperature. It also is apparent that the changes produced are not the same for each type of rubber and asphalt.

Effect on Ductility

Ductility tests were made on the original asphalts and the various rubber-asphalt blends after heating at 300° F. for 2 hours, as shown in tables 5 and 7. In the preliminary study ductility tests were made at both 60° and 77° F., while in the more detailed study ductility tests were made at 77° F. only.

In general, the addition of rubber to the asphalt caused a reduction in ductility at 77° F. The exceptions to this were the blends of asphalt AC–3 containing 5.0, 7.5, and 10.0 percent of rubber R–1 and the same asphalt with 5.0 and 7.5 percent of rubber R–13. It is probable that the ductility of many of the blends was affected by the presence of free particles of rubber. For those blends that had little or no decrease in ductility, the rubber was more completely dispersed. For the blends of rubber R 1 and R–13 with asphalt AC–3, the ductility was indeterminable except for the 10-percent blend of rubber R–13 due to the fact that their ductility was beyond the capacity of the ductility machine. Figure 8 shows the effect of rubber content on the ductility of some of the rubber-asphalt blends.

In the studies of paving asphalts previously referred to (*18*), when 50–60 and 85–100 penetration asphalts were subjected to lower temperatures, the values for ductility decreased, and at some definite temperature for the particular type of asphalt it became zero. For instance, the ductility of 50–60 penetration asphalts when tested at 77° F. at 5 cm. per minute, had ductilities of from 27 to 250+ cm. with an average of 175.5. The ductility of 85–100 penetration asphalts under the same conditions had ductilities of 101 to 250+ cm. with an average of 179.8. At 39.2° F., under a pull of 5 cm. per minute, the ductility of the same asphalts varied from 0 to 4.0 cm. with an average of 1.6 cm. for the 50–60 grade and from 0 to 9.7 cm. with an average of 4.3 cm. for the 85–100 grade. Compared with paving asphalts, blown asphalts have a low rate of change in ductility with change in temperature.

Low-Temperature Ductility

Ductilities of the asphalts and rubber-asphalt blends containing 7.5 percent of the various rubbers were determined at 39.2°, 60°, and 77° F. and the results are given in table 10. The effect of temperature on ductility for the original asphalts, the rubber-asphalt blends, and a typical blown asphalt included for comparison is shown in figure 9.

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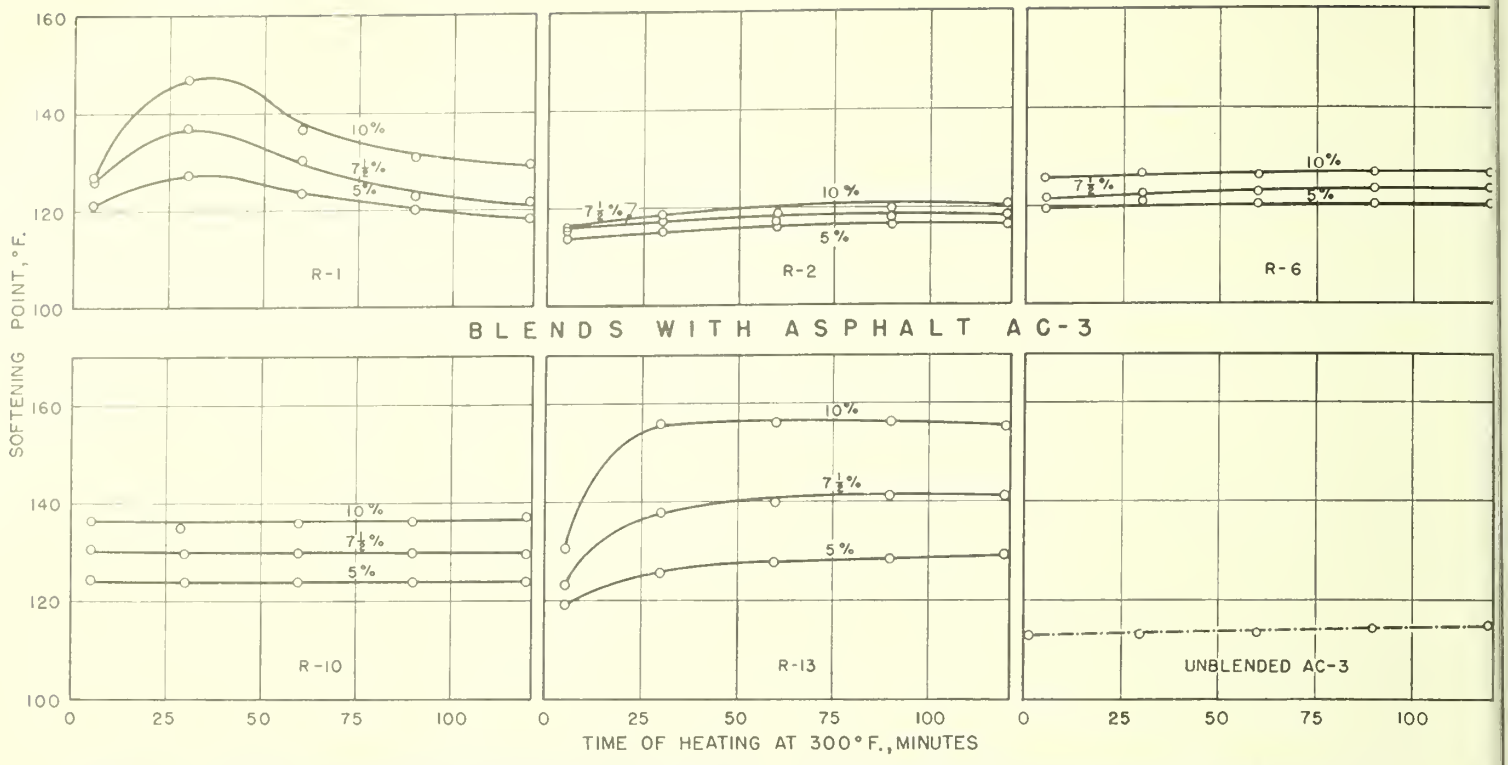

Figure 4.—Change in softening point during heating of blends of asphalt AC–3 with various amounts of powdered rubbers.

In the majority of cases, the rubber-asphalt blends did not change in ductility with change in temperature when tested at 5 cm. per minute as slowly as the typical blown asphalt also shown in figure 9, or as rapidly as the unblended petroleum asphalts. However, the ductility of the blends of rubbers R–7 and R–13 with asphalt AC–1 and of rubber R–13 with asphalt AC–2 increased as the temperature of test was lowered, and at 39.2° F. these materials had ductilities much higher than many normal asphalts at temperatures producing maximum ductility. While the unblended asphalt AC–3 had no ductility at 39.2° F., the rubber-blend with 7.5 percent of R–1 rubber had a ductility of 70 cm. The addition of 7.5 percent R–13 rubber to AC–3 gave ductilities greater than the capacity of the testing machine at all test temperatures. This is a most unusual condition and it indicates that natural rubber with asphalt AC–3, and the GR–S type II rubber with all asphalts, definitely improve the low-temperature ductility characteristics of the asphalts covered in this study.

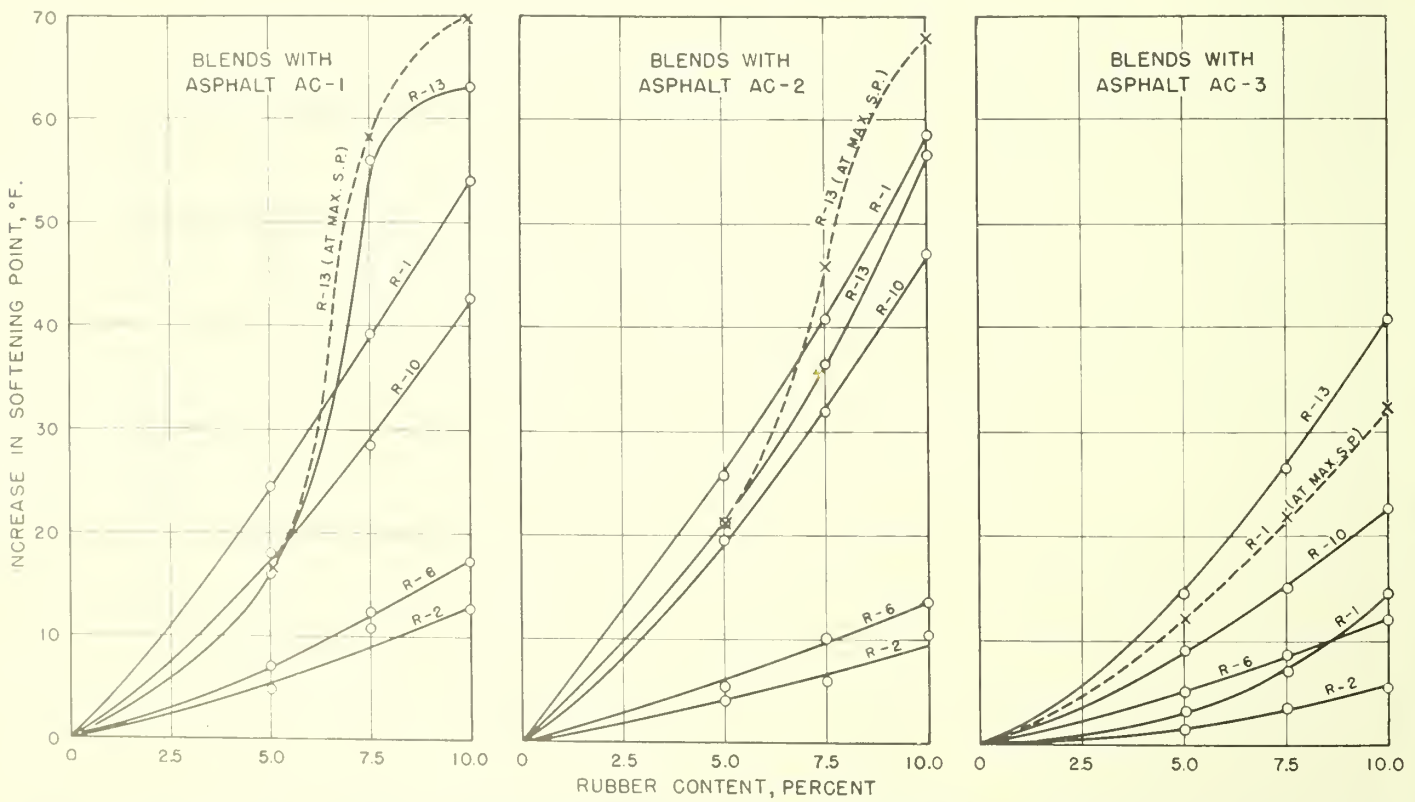

Figure 5.—Effect of rubber content on increase in softening point of rubber-asphalt blends (curves are for values after 2-hour mixing except as noted).

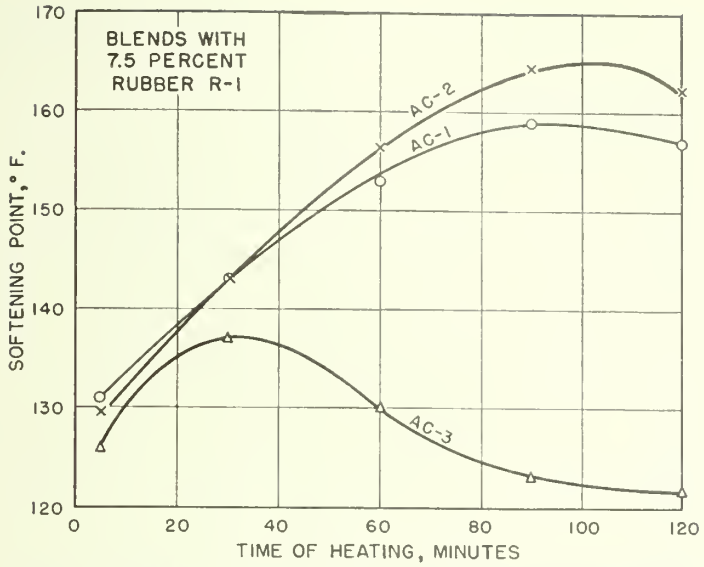

Figure 6.—Effect of time of heating on the softening point of rubber-asphalt blends prepared with different asphalts and 7.5 percent rubber R–1.

Rebound After Ductility

During the preparation and the handling of the various rubber-asphalt blends, it was noted that there was considerable difference in their elastic properties. The blends containing rubbers R–1 and R–13 particularly showed ability to recover to their original shape when deformed. In order to get some measure of this elastic behavior, the following data was obtained from the ductility test.

After the material was elongated to the point of rupture in the ductility test, the elongated specimen was allowed to remain undisturbed for 15 minutes and the length of the broken threads was measured. The amount of original elongation less the length after 15 minutes was termed the rebound. This rebound in centimeters was calculated as a percentage of the original elongation. These values are given in tables 5 and 7 for the original blends and for the residues of the blends from the thin-film oven test.

Due to the high ductility values and to the fineness of the broken thread, the rebound of unblended asphalts AC–1, AC–2, and AC–3, all blends of AC–3 containing rubber R–1, and the 5- and 7.5-percent blends of rubber R–13 and AC–3 could not be determined. The data in table 7 show, in general, that for the blends of asphalts AC–1 and AC–2 with various amounts of rubber, those containing rubbers

Table 9.—Penetration of asphalts and rubber-asphalt blends at 32° and 100° F. (extrapolated from log-penetration-temperature curves)

Rubber in blend	Penetration, 100 g., 5 sec.—					
	At 32° F.			At 100° F.		
	AC–1	AC–2	AC–3	AC–1	AC–2	AC–3
None	9.0	8.0	4.8	275	295	365
R–1, 5 percent	10.3	12.0	[1] 6.8	170	160	363
R–1, 7.5 percent	9.6	12.0	[1] 8.6	140	138	358
R–1, 10 percent	9.4	10.8	[1] 17.3	119	125	330
R–2, 5 percent	8.2	9.3	5.4	240	235	330
R–2, 7.5 percent	8.3	9.6	6.1	196	218	293
R–2, 10 percent	8.4	10.2	7.2	176	180	263
R–6, 5 percent	9.0	9.2	5.3	198	215	262
R–6, 7.5 percent	8.5	8.4	4.9	182	190	241
R–6, 10 percent	7.6	9.0	5.6	160	164	217
R–10, 5 percent	11.3	15.8	6.8	180	167	261
R–10, 7.5 percent	14.0	18.3	8.2	138	138	227
R 10, 10 percent	18.7	21.6	11.0	126	121	197
R–13, 5 percent	10.5	13.9	[1] 8.0	195	150	282
R–13, 7.5 percent	14.5	16.4	[1] 8.5	142	127	260
R–13, 10 percent	17.3	19.0	[1] 11.0	127	106	227

[1] Values extrapolated from curves through penetrations at 77° and 95° F.

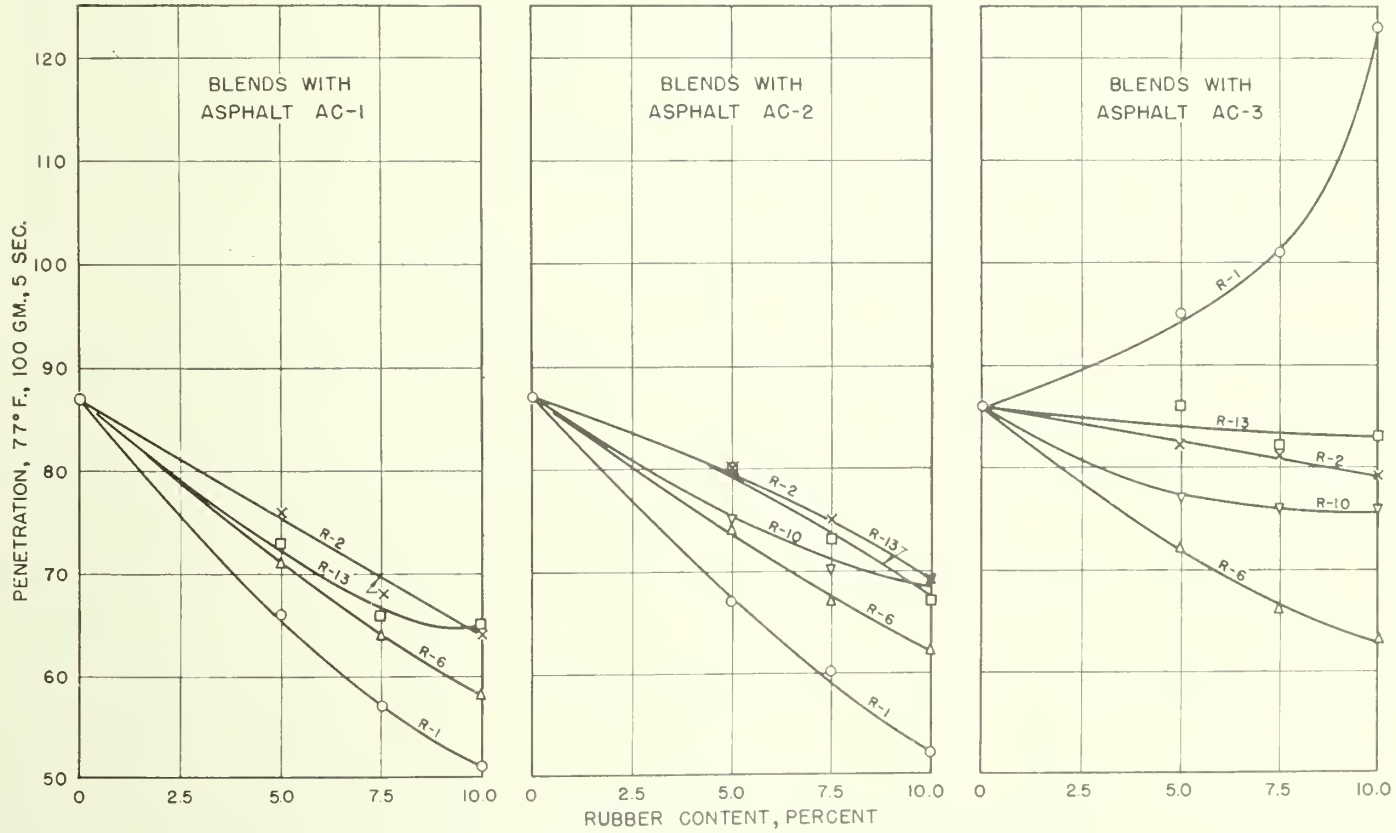

Figure 7.—Effect of rubber content on penetration of rubber-asphalt blends.

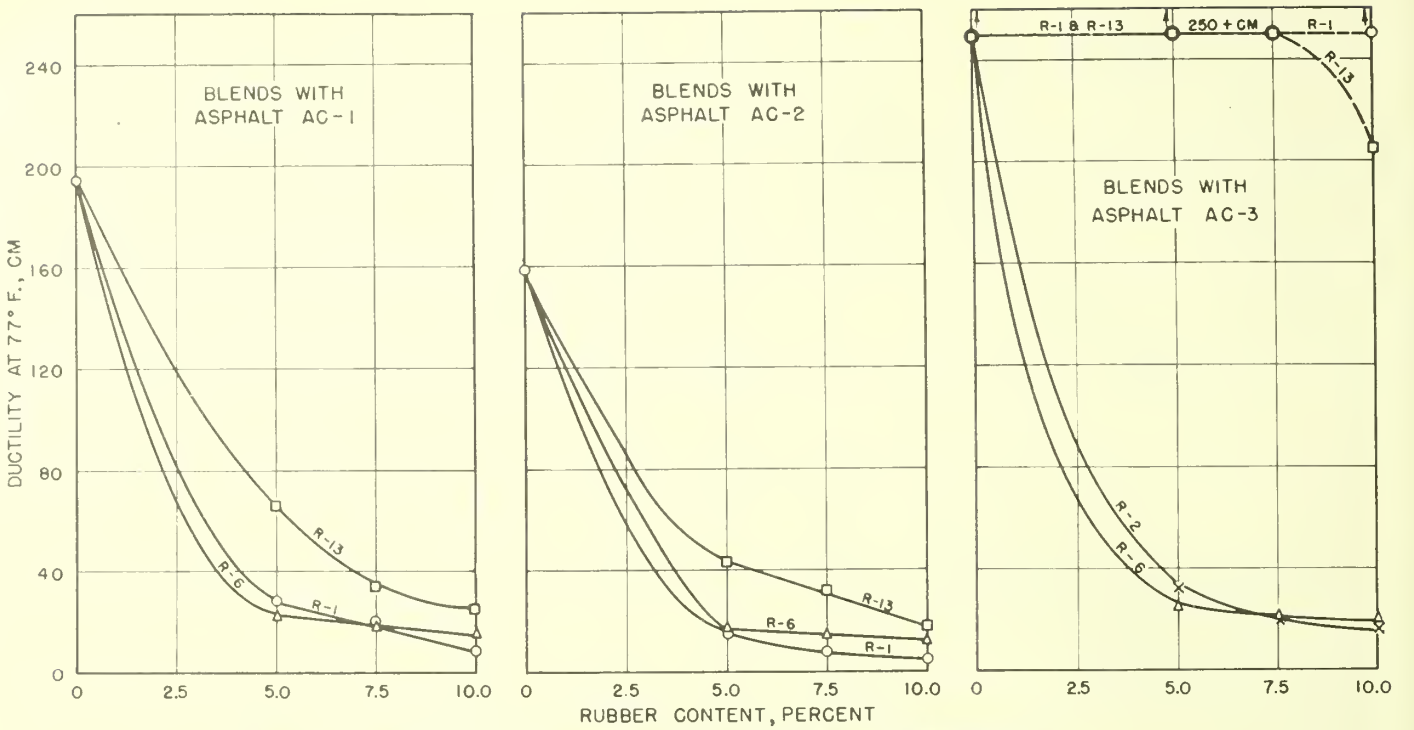

Figure 8.—Effect of rubber content on ductility of rubber-asphalt blends.

R 1 and R–13 had the highest amount of rebound and those containing R–2 and R–6 the lowest rebound.

Elasticity Measured

The development or increase in elasticity of asphaltic materials has been considered as one of the more important changes induced by the addition of rubber. Such data as those obtained by measuring the rebound of the broken thread after the ductility test no doubt indicates this elastic property. However, this test is limited generally to relatively short elongations, that is, those less than 100 cm. Since some of the asphalts and rubber-asphalt blends had elongations well over 100, the rebound could not be determined. Therefore, another test was developed in the laboratory by which the difference in elastic properties could be measured.

In this test a determination was made of the time required for a steel cylinder 1 inch in diameter and weighing 200 grams to penetrate into the asphalts or rubber-asphalt blends to a

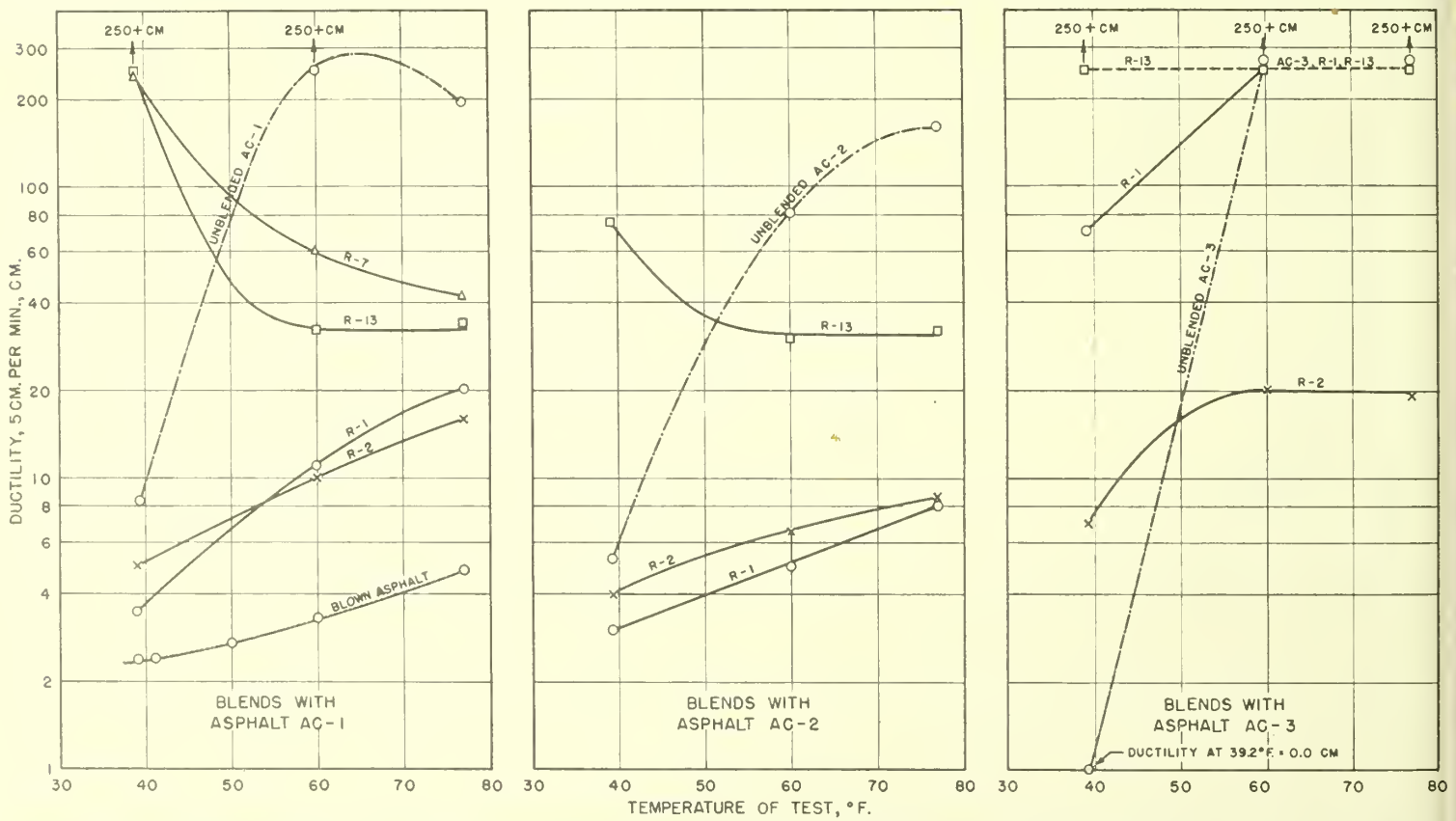

Figure 9.—Ductility-temperature curves for unblended asphalts and rubber-asphalt blends containing 7.5 percent of various powdered rubbers.

Table 10.—Results of ductility tests on asphalts and blends containing 7.5 percent rubber

Blend identification	Ductility at—			
	5 cm. per min. at—			1 cm. per min. at 39.2° F.
	77° F.	60° F.	39.2° F.	
	Cm.	*Cm.*	*Cm.*	*Cm.*
Unblended AC–1	195.0	+250	8.5	32.0
Blend of AC–1 and 7.5 percent of—				
R–1	20.0	11.0	3.5	3.5
R–2	16.0	10.0	5.0	8.0
R–6	19.0	9.5	4.5	6.0
R–7	42.0	60.0	239.0	123.0
R–10	18.0	8.0	2.5	3.3
R–13	34.0	32.0	+250	110.0
R–14	19.0	9.0	3.0	3.0
Unblended AC–2	159.0	80.0	5.3	8.0
Blend of AC–2 and 7.5 percent of—				
R–1	8.0	5.0	3.0	3.5
R–2	8.5	6.5	4.0	5.2
R–6	14.5	6.0	3.5	3.8
R–10	7.8	5.0	2.3	2.5
R–13	32.0	30.0	75.0	22.0
Unblended AC–3	+250	+250	0	+250
Blend of AC–3 and 7.5 percent of—				
R–1	+250	+250	70.0	158.0
R–2	19.0	20.0	7.0	15.5
R–6	21.0	22.0	1.0	13.0
R–10	36.0	36.0	7.0	20.0
R–13	+250	+250	+250	+250

total depth of 0.6 inch under a superimposed load of 10 pounds. Immediately after the cylinder had penetrated the material to a depth of 0.6 inch, the 10-pound load was removed and change in the depth of penetration of the cylinder was determined at 1-minute intervals for a total of 5 minutes. The results of these tests are given in table 11.

A plot of the depth of penetration against the time of penetration, both on logarithmic scales, resulted in linear relation for the unblended asphalt and for various rubber-asphalt blends. This relation is shown in figure 10 for asphalt AC–1 and the blends containing 5 and 10 percent of rubbers R–1, R–2, R–6, R–10, and R–13. Although the straight lines are approximately parallel, they show the wide effect of the various amounts and types of rubber on the resistance to penetration of the cylinder. For instance, it took 38 times as long for the cylinder to penetrate 0.6 inch in the blend containing 10 percent of rubber R–1 as it did in the asphalt itself. Blends containing rubber R–2 showed the least resistance to penetration, the time of penetration of the 10 percent blend being only 2.6 times that of the asphalt.

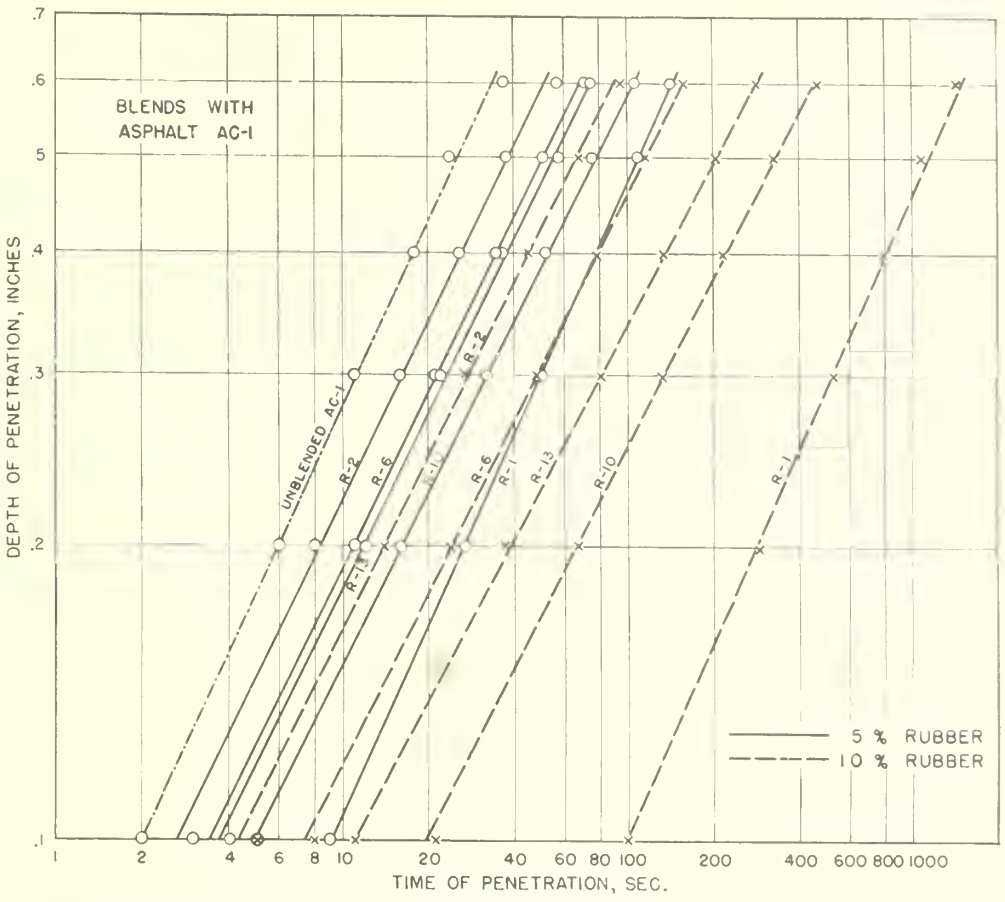

Figure 10.—Effect of time on penetration of a 1-inch, 200-gram cylinder under a 10-pound load into blends of asphalt AC–1 and various powdered rubbers (tested at 77° F.).

In figure 11 the change in depth of the cylinder after removing the 10-pound load is plotted against the time of recovery for asphalt AC–1 and the blends containing 5, 7.5, and 10 percent of the various rubbers. Although the data for the blend containing 7.5 percent of R–13 were incomplete, the curve is assumed as shown by the broken line. For the asphalt, the weight of the cylinder was sufficient to cause continued penetration while for all the rubber-asphalt blends there was an immediate recovery or rebound, the extent and duration of which varied with the type and amount of rubber in the blend. For the 5-percent rubber blends, this recovery was only temporary and was followed by a renewed settlement of the plunger at some period within the 5-minute interval except for the R–1 blend which remained in a static position. For the blends containing 7.5 and 10 percent of rubbers R–1, R–10, and R–13, the recovery increased during the entire 5 minutes, while for blends containing the same amount of rubbers R–2 and R–6 the recovery decreased after the 1- or 2-minute periods.

This study shows that there was a wide difference in the amount of recovery of the cylinder which further indicates the varying degree of elasticity imparted by the various rubbers. It is apparent that rubbers R–1, R–10, and R–13 impart a considerable amount of elasticity to the asphalt.

Table 11.—Results of special penetration and recovery tests [1] on asphalt AC–1 and rubber-asphalt blends

Blend identification	Time for loaded cylinder to penetrate—						Recovery of unloaded cylinder after—				
	0.1 inch	0.2 inch	0.3 inch	0.4 inch	0.5 inch	0.6 inch	1 minute	2 minutes	3 minutes	4 minutes	5 minutes
	Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*0.001 in.*	*0.001 in.*	*0.001 in.*	*0.001 in.*	*0.001 in.*
Unblended AC–1	2	6	11	18	24	37	−16	−30	−47	−59	−75
Blend of AC–1 and—											
R–1, 5 percent	9	27	50	78	108	142	45	53	57	57	57
R–1, 7.5 percent	38	116	215	324	447	580	30	38	44	46	49
R–1, 10 percent	99	296	535	800	1,090	1,413	30	37	40	44	47
R–2, 5 percent	3	8	16	26	38	57	18	10	3	−15	−28
R–2, 7.5 percent	4	13	25	43	65	94	33	34	31	27	23
R–2, 10 percent	5	14	27	45	68	95	44	45	41	38	33
R–6, 5 percent	4	11	21	35	51	71	10	15	7	−1	−10
R–6, 7.5 percent	6	17	32	53	78	109	32	29	27	22	17
R–6, 10 percent	8	24	48	78	115	159	33	34	33	30	27
R–10, 5 percent	5	16	32	52	76	106	38	40	40	37	33
R–10, 7.5 percent	12	35	70	113	168	235	43	56	58	61	64
R–10, 10 percent	21	67	133	219	327	462	41	51	58	64	67
R–13, 5 percent	4	12	22	36	54	75	40	43	42	39	30
R–13, 7.5 percent	8	24	49	83	125	174					62
R–13, 10 percent	11	39	81	135	204	287	60	74	81	88	91

[1] Penetration is that of a 1-inch diameter, 200-gram cylinder, loaded with a 10-pound weight. Recovery is determined after removal of the 10-pound weight. Tests were made at 77° F.

Effect on Flow of Asphalts

It has been stated by other investigators that the addition of rubber to asphalts changes the flow properties and thereby increases the resistance to distortion of paving mixtures containing this type of binder under high temperatures and heavy traffic. Endres, et

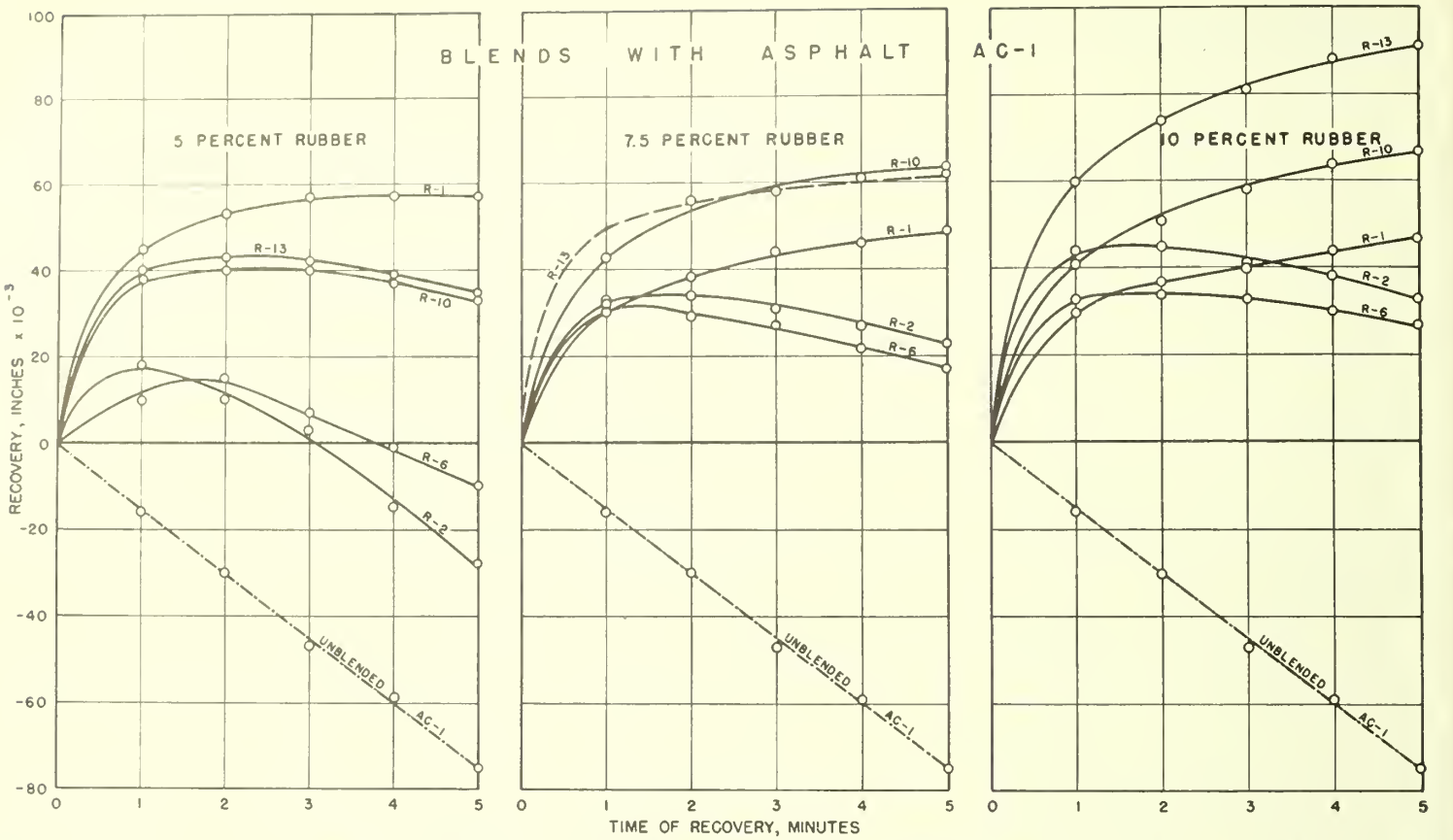

Figure 11.—Recovery of 1-inch, 200-gram cylinder after removing 10-pound load, for blends of asphalt AC–1 and various powdered rubbers.

al., (*12*) have shown that the addition of rubber affects the flow properties of asphalts and the amount of flow varies with the different types of rubber.

The method used in this study for determining the flow was similar to the method used for measuring the flow of blown asphalts (*20*). In this test the material was heated to a fluid condition and poured into an amalgamated brass mold, ⅜ inch in inside diameter and ¾ inch in height. After cooling, the excess material was cut from the top and the cylindrical specimen removed. This specimen was then placed on a corrugated brass plate, supported at an angle of 45 degrees with the lower end of the cylinder on a line 15 cm. above the end of the plate. The apparatus is shown in figure 12. For blown asphalts the amount of flow is determined by heating the specimen at 150° F. for 4 hours but, due to the much higher rate of flow of the rubber-asphalt blends studied here, it was necessary to modify the temperature and time of heating. It was found that all the mixtures were within the range of 15 cm. when tested at 140° F. for 1 hour. The results of the flow test under these conditions are given in tables 5 and 7.

The results of the tests show that, except for the blend of asphalt AC–1 with 5 percent of rubber R–15, flow of all the blends containing rubber was less than the flow of the unblended asphalts. For the same rubber content the amount of flow varied with the different asphalts and rubber used. For the same rubber and asphalt, the amount of flow decreased as the rubber content was increased.

In order to show the relation between flow and softening point of the various mixtures, the softening point was plotted against the log of the flow for the three unblended asphalts and five of the rubber-asphalt blends, as shown in figure 13. Although there is some spread among the points, the average curves for the three asphalts are almost identical. In general, the relation between softening point and flow of the rubber-asphalt blends was similar to that found in testing rubber-asphalt compounds used for sealing concrete expansion joints (*19*). In this previous work it was found that the rate of flow of the joint sealing materials was proportional to their softening points and it was recommended that the softening point determination be used to evaluate the flow properties of these materials instead of the flow test.

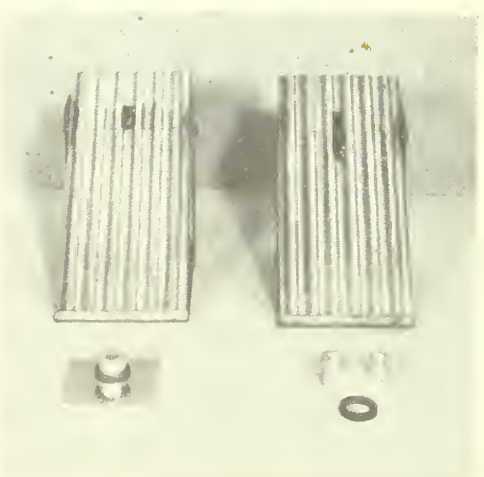

Figure 12.—Apparatus used in determining flow.

The flow test is a measure of the susceptibility to temperature change of the material as can be seen in figure 14 where the slope of the log-penetration-temperature curve is plotted against the log-flow of the blends prepared with asphalt AC–2 and the various rubbers. Except for three blends the points fall close to a straight line.

Thin-Film Oven Tests

For several years the thin-film oven test (*21, 22*) has been advocated as a means of measuring the amount of change that may occur in asphalt cements when mixed with an aggregate at high temperature. This test is more severe than the standard test for loss on heating due to the larger surface area of the test sample that is exposed. The thin-film oven test is made by exposing a ⅛-inch film of asphalt in an aluminum pan 5.5 inches in diameter and ⅜ inch in height in an oven at 325° F. for 5 hours. The loss in weight is determined and the residue tested for penetration, ductility, and softening point. The results of the thin-film oven test on the rubber-asphalt blends are given in tables 5 and 7.

In general, when asphalt cements are exposed to the thin-film oven test, there is a slight loss in weight and a varying decrease in penetration and ductility and increase in softening point. For asphalts containing rubber, this was not found to be true for all the blends tested. As compared with the original blends, the thin-film residues of asphalts AC–1 and AC–2 containing the three percentages of rubber R–1 had approximately the same or higher penetration, much higher ductility, and much lower softening point. The thin-film

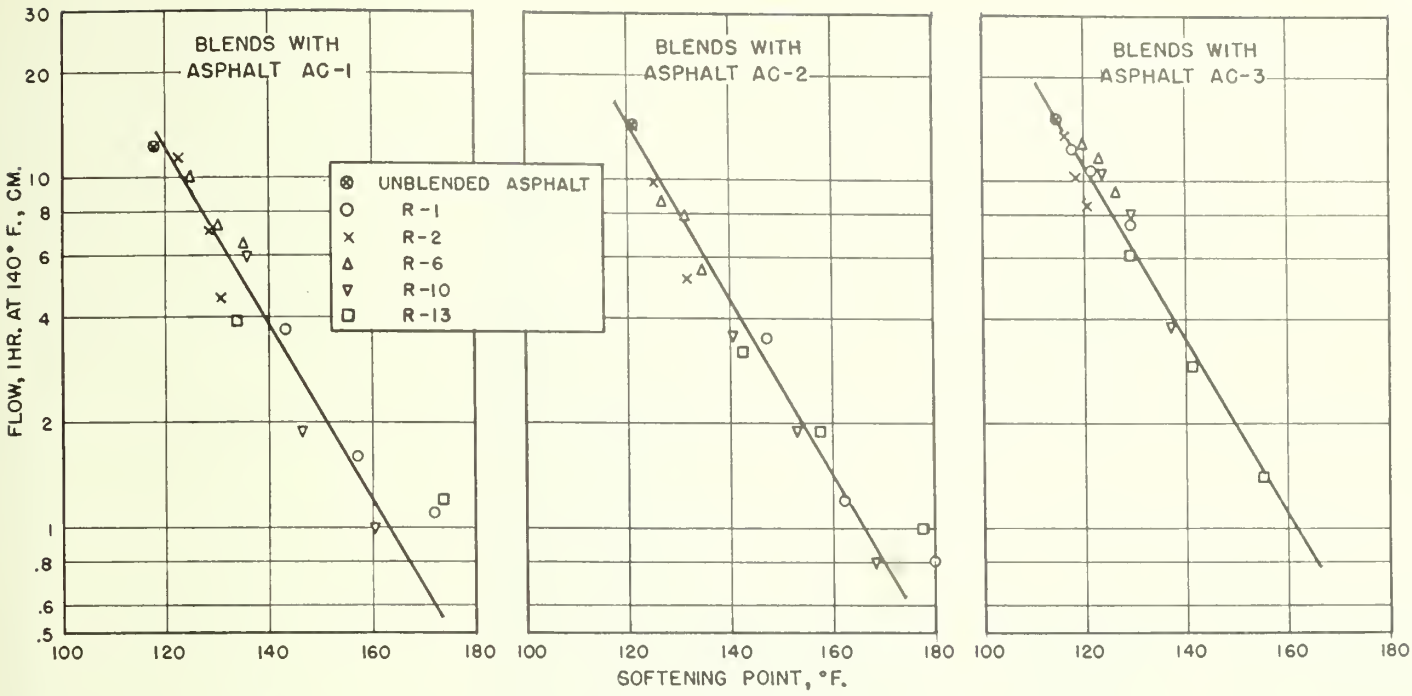

Figure 13.—Relation between softening point and flow of rubber-asphalt blends.

residues of asphalt AC–1 with 7.5 percent rubber R–13 and asphalt AC–3 with 7.5 and 10 percent rubber R–1 had lower penetrations and somewhat lower softening points than did the corresponding original blends. The ductility of the thin-film residues from blends made with the three asphalts with rubbers R–2 and R–6 was approximately the same or was lower than the ductility of the corresponding original blends. The ductility of all the residues containing rubber R–10 was higher than, or approximately the same, as the ductility of the corresponding original blends.

The results of the thin-film oven tests indicate that there could be a wide difference in the changes which would take place in the properties of the various rubber-asphalt blends when subjected to high temperatures, such as those encountered in the processing of hot mixtures.

Solubility in Carbon Disulfide

In order to obtain some information on the blending action of the various rubbers in asphalts, the test for solubility in carbon disulfide was made on the various rubber-asphalt blends. Due to the inability to filter some of the blends after dissolving with carbon disulfide, it was necessary to reduce the size of sample to approximately 0.5 gram and also use celite as a filter aid. Even with these aids the blends containing rubbers R–1 and R–13 were extremely slow to filter and results of tests showed poor reproducibility. The results of the tests for solubility in carbon disulfide are given in tables 5 and 7.

The results show a wide difference in the solubility of the blends prepared with different rubbers. In general, it can be seen that those rubbers (R–1 and R–13) that produced the greatest changes in the physical characteristics of the asphalts were also the most soluble in carbon disulfide. Since in no case is the added rubber entirely soluble in carbon disulfides, it is indicated that the characteristics of the asphalts extracted and recovered from paving mixtures containing rubber may differ greatly from the asphalt blends as they actually exist in the pavement.

Effect of Heating Time on Viscosity

In order to obtain additional information on the changes that occur during the mixing of the rubbers with the asphalts, viscosity tests were made on blends containing 5 percent rubber heated at 300° F. for 4 hours. During this period of heating, viscosity determinations were made every 30 minutes with a Brookfield Syncho-Lectric viscometer, shown in figure 15.

The principle of this instrument is the measurement of the drag produced on a spindle rotating at a constant speed while immersed in the material being tested. The amount of drag is indicated by a pointer attached to the spindle shaft which measures the tension produced in a spiral spring connected between the shaft and the outside of the dial which is connected to the motor shaft. The range in viscosity is obtained with a

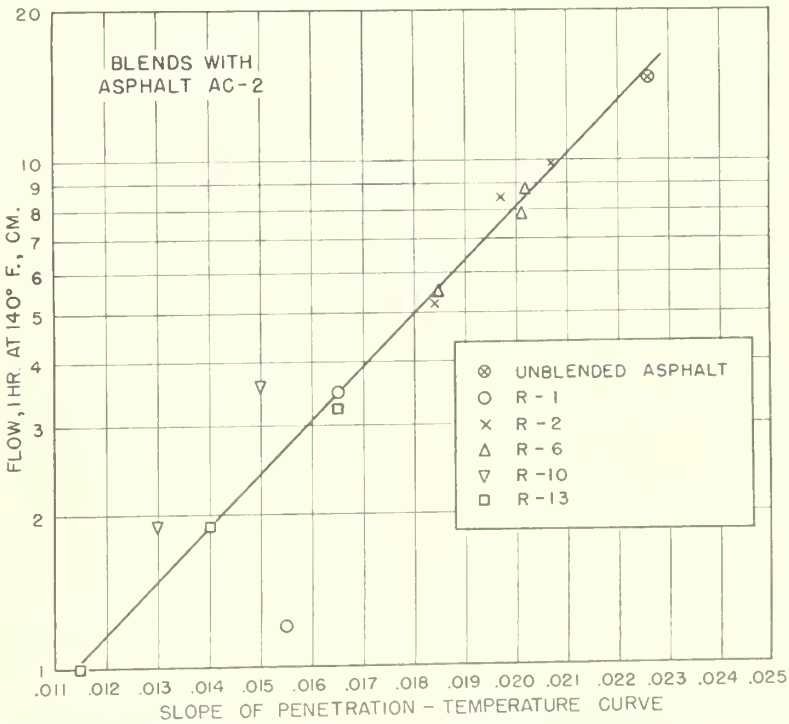

Figure 14.—Relation of slope of log-penetration-temperature curve to the flow of blends of asphalt AC–2 and various powdered rubbers.

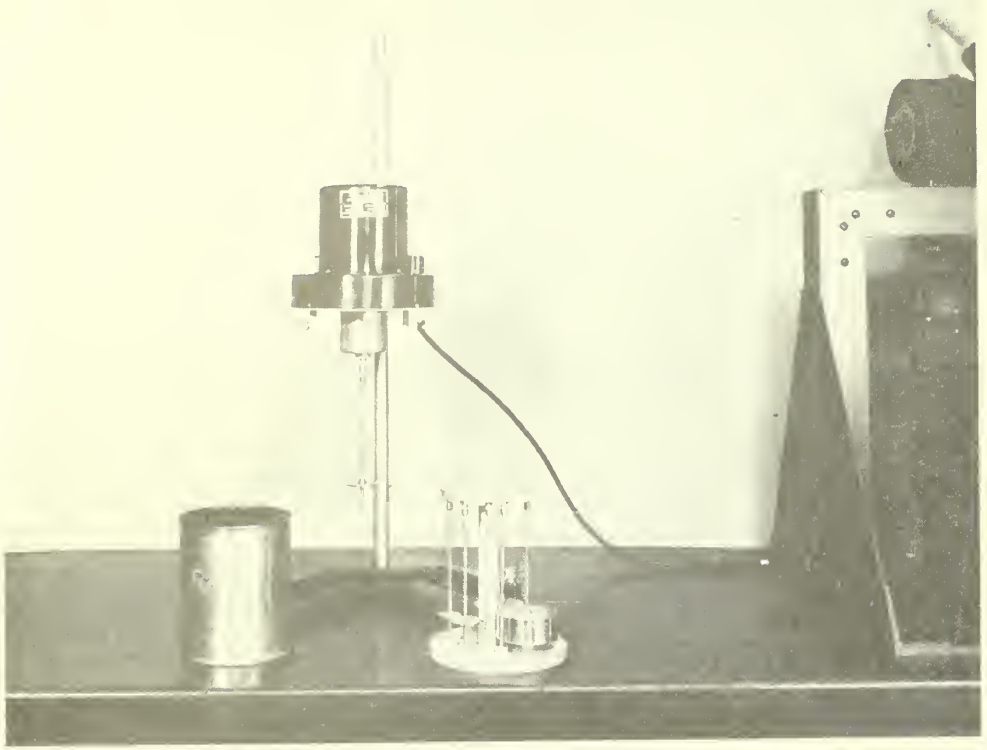

Figure 15.—Synchro-Lectric viscometer used to determine viscosity of rubber-asphalt blends.

series of spindles, each covering a part of the range. Viscosity is calculated by multiplying the reading on the dial by a factor, depending upon the spindle and speed of rotation used. The results of the viscosity determinations are given in table 12 for the blends prepared with each of the three asphalts and 5 percent of each of the rubbers. The effect of the time of heating on the viscosities of these blends is shown in figure 16.

There was a very wide range in the change in viscosity of the various mixtures during the 4-hour period. For all asphalts, blends with rubbers R–1 and R–13 developed the highest viscosities and blends with rubbers R–2 and R–6 the lowest viscosities. Considering the maximum viscosity reached during the 4 hours, the order of the five rubbers is the same for the three asphalts and this order is the same as was found for the increase in softening point. Here also the change in viscosity was effected by the source of the asphalt.

Viscosity-Temperature Relations

Consistency is a property of asphaltic road materials that is of extreme importance from the standpoint of their application to road surfaces or in plant mixing with an aggregate. Since it was evident from earlier work that there was a wide difference in the consistency of the various rubber-asphalt blends, the viscosity-temperature relations of blends of asphalt AC–1 with 5 percent of rubbers R–1, R–2, R–6, R–7, and R–13 were determined. The blends were prepared by mixing the rubber with the asphalt at 300° F. for 2 hours. At the end of the period, the mixing paddle was removed and viscosities determined on the material with the Brookfield viscometer, described earlier in this report.

Viscosity determinations were made first at 300° F. and then at lower temperatures as the material cooled. In order to reduce the time required for testing, tests were not made at predetermined temperatures. For viscosity determinations above 309° F., the blends were reheated after the test at the lowest temperature had been completed. It was apparent from the results obtained that very little change in viscosity occurred in the blends during the time they were maintained at these elevated temperatures.

The results of the viscosity determinations of the various rubber-asphalt blends over a range in temperature are given in table 13. These results are plotted in figure 17 on a log-temperature log-viscosity basis. By plotting in this manner, straight-line relations were obtained.

The viscosity-temperature relations show a very wide difference in the viscosities of blends containing the various rubbers over the entire range in temperatures. As was indicated previously, blends with rubbers R–1 and R–13 produced the greatest change in viscosity and blends with rubbers R–2 and R–6 the least change. All blends had viscosities well above the viscosity of the unblended asphalt.

In order to compare these viscosity values with Saybolt Furol viscosity, which is more commonly used, an approximate conversion of absolute viscosity to Furol viscosity can be obtained from the standard method of conversion of kinematic viscosity to Saybolt Furol viscosity (ASTM designation D 666–44) as follows:

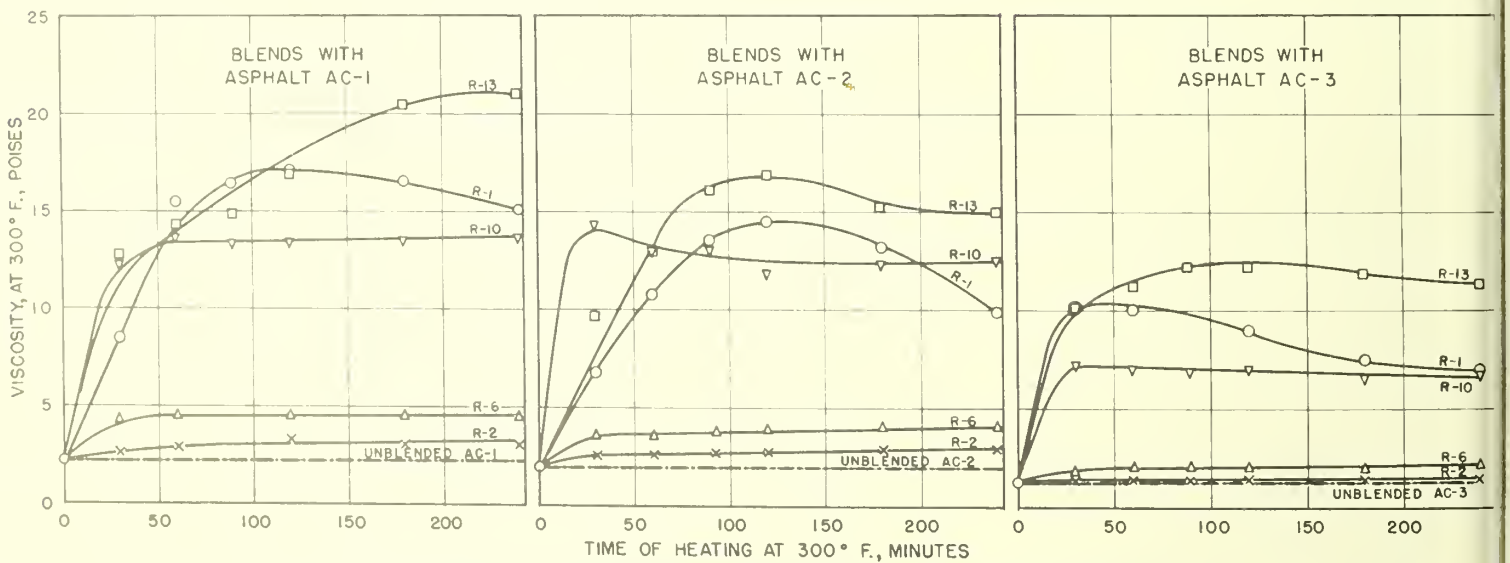

Figure 16.—Effect of time of heating on viscosity of 5 percent rubber-asphalt blends.

Table 12.—Effect of time of mixing at 300° F. on viscosity at 300° F. of asphalts and blends containing 5 percent rubber

Blend identification	Viscosity after heating and mixing for—						
	0 min.	30 min.	60 min.	90 min.	120 min.	180 min.	240 min.
	Poises	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*
Unblended AC–1	2.37						
Blend of AC–1 and 5 percent of—							
R–1		8.50	15.55	16.40	17.10	16.55	15.20
R–2		2.65	2.90		3.30	3.10	3.10
R–6		4.35	4.50		4.50	4.55	4.60
R–10		12.38	13.70	13.35	13.30	13.55	13.75
R–13		12.85	14.35	14.90	16.95	20.50	21.05
Unblended AC–2	2.00						
Blend of AC–2 and 5 percent of—							
R–1		6.85	10.80	13.55	14.60	13.20	10.00
R–2		2.65	2.63	2.70	2.75	2.90	2.90
R–6		3.70	3.65	3.80	3.95	4.10	4.10
R–10		14.35	13.00	13.10	11.85	12.40	12.55
R–13		9.65	12.95	16.20	16.90	15.30	15.05
Unblended AC–3	1.25						
Blend of AC–3 and 5 percent of—							
R–1		10.20	10.10		9.05	7.50	6.95
R–2		1.30	1.35	1.35	1.35	1.40	1.40
R–6		1.90	2.05	2.15	2.10	2.05	2.15
R–10		7.30	7.10	6.90	7.10	6.60	6.65
R–13		10.05	11.25	12.30	12.25	11.90	11.40

Saybolt Furol viscosity $= V_{cs} \times 0.470$

where V_{cs} is the viscosity in centistokes.

Also: $$V_{cs} = \frac{V_p}{\text{density}} \times 100$$

where V_p is the viscosity in poises.

Since the density of the asphalts and rubber-asphalt blends is approximately 1.00,

$$V_{cs} = V_p \times 100.$$

Thus the viscosity data in poises as given in table 13 and shown in figure 17 can be converted to Furol viscosity, as follows:

Furol viscosity $= V_p \times 47.0$.

For plant mixing of hot asphaltic concrete, a Furol viscosity range of 75 to 150 seconds has been considered as optimum. Converted by the equation just derived, the optimum is 1.60 to 3.19 poises. This range is indicated in figure 17, and it will be noted that the temperatures at which the asphalt and rubber-asphalt blends have this optimum viscosity are as follows:

Unblended asphalt AC–1	285–305° F.
AC–1 with 5 percent R–2	300–325° F.
AC–1 with 5 percent R–6	315–340° F.
AC–1 with 5 percent R–7	320–350° F.
AC–1 with 5 percent R–1	355–380° F.
AC–1 with 5 percent R–13	370–395° F.

The temperature ranges for use in plant mixing the different rubber-asphalt blends are indicated in figure 17.

The use of preblended rubber asphalts may not only require higher mixing temperatures but also some additional equipment to handle the more viscous binder. In the Massachusetts project, previously referred to (*9*), the GR–S synthetic rubber powder was prepared from standard GR–S synthetic rubber latex in such a manner as to produce a dry powder at least 50 percent of which passed the No. 40 sieve and 100 percent the No. 20 sieve. No information was given as to the degree of vulcanization of the synthetic latex. The rubber-asphalt blend was prepared at the plant of the asphalt producer and the very viscous condition of the rubberized blend presented a handling problem as additional pumps were necessary to circulate the rubberized asphalt in the distributor during transportation of the material to the storage tank at the paving plant and to provide satisfactory flow between the plant storage tank and the mixer box.

Road construction practice has shown that the selection of the proper temperature of application of bituminous materials to aggregate, either on the road or in a mixing plant, to give proper coating of the aggregate and workability of the mixture, is dependent upon the consistency of the material at the time of application or mixing. For plant mixing this consistency depends to a large extent upon the temperature of the aggregate. For the materials studied here it would appear that

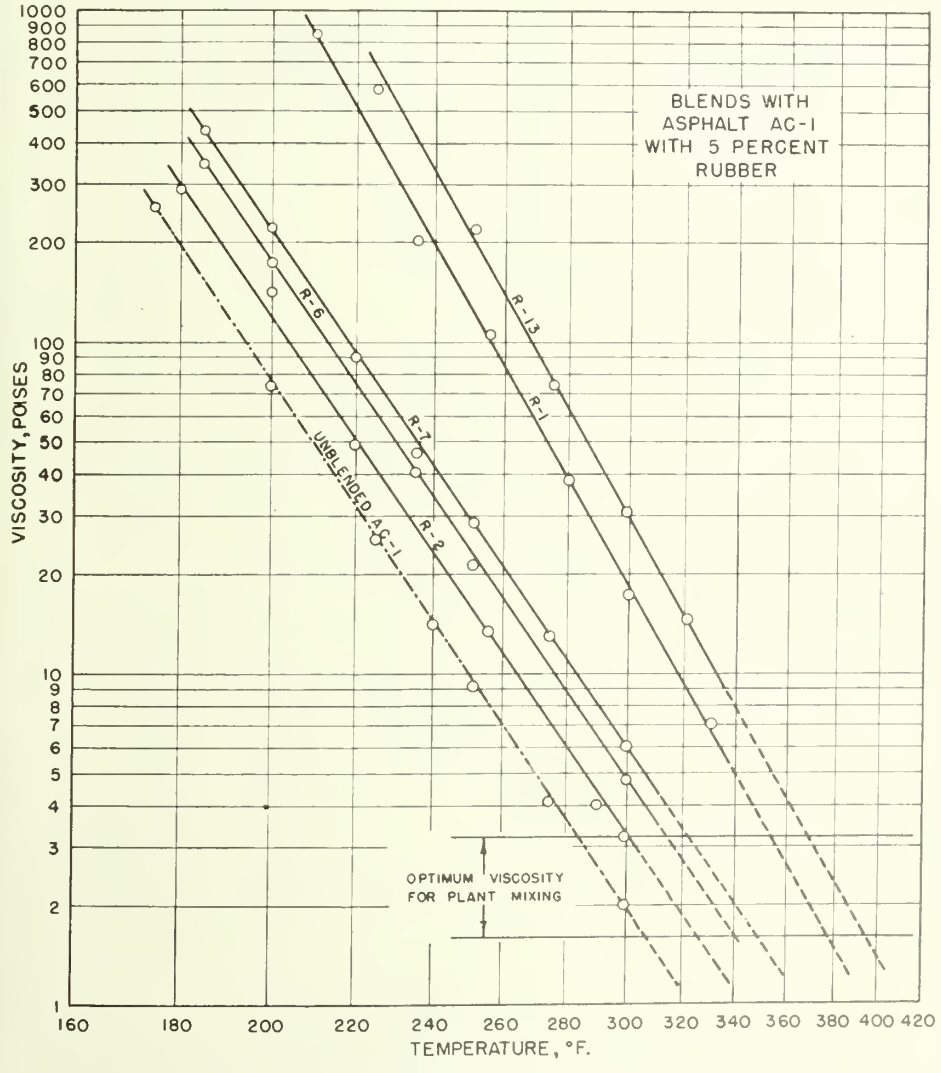

Figure 17.—Viscosity-temperature relation of rubber-asphalt blends.

Table 13.—Viscosity of asphalt AC–1 and rubber-asphalt blends at various temperatures [1]

Test temperature	Viscosity of blend of asphalt AC–1 and 5 percent of rubber indicated					
	None	R–1	R–2	R–6	R–7	R–13
° *F.*	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*
162				630.0		
175	256.5					
180			287.5			
185		1,800.0		353.0	433.5	
200	74.0		140.8	177.0	220.0	1,400.0
210		840.0				
220			48.8		88.8	
225	25.4					577.0
235		200.0		40.4	46.0	
240	14.0					
250	9.2			21.2	28.5	216.0
255		104.0	13.4			
275	4.1				12.9	74.0
280		38.0				
290			4.0			
300	2.0	17.1	3.2	4.8	6.0	30.4
320						14.4
330		7.0				

[1] Blends prepared by heating at 300° F. for 2 hours.

Table 14.—Results of tests on blends of asphalt AC–1 and 5 percent of various rubbers mixed at various temperatures

	Blend with R–1, mixed at			Blend with R–2, mixed at			Blend with R–6, mixed at—			Blend with R–13, mixed at—		
	300° F.	338° F.	425° F.	300° F.	375° F.	425° F.	300° F.	375° F.	425° F.	300° F.	338° F.	425° F.
Mixing time, hr.	2	2	2	4	4	4	4	4	4	2	2	2
Softening point (ball-in-ring):												
5 min., °F.	127.0		133.3	117.0			125.2			127.2	130.5	133.8
30 min., do	133.5	142.3	134.3	122.7	120.0	124.0	124.9	128.3	129.0	132.0	138.0	144.0
60 min., do	130.0	140.5	132.8	124.5	122.1	125.0	127.0	129.0	130.0	134.0	139.1	142.0
90 min., do	133.0	137.7	130.0	121.3	123.5	125.0	125.6		131.5	134.3	141.4	149.0
120 min., do	132.5	137.3	128.8	122.9	124.3	126.3	125.0	129.8	132.5	134.0	141.7	152.1
240 min., do				123.8	127.3	129.9	126.5	132.4	136.4			
Penetration:												
At 50° F.	22	20	18	18	17	15	16	16	15	22	23	17
At 77° F.	66	69	65	73	66	63	66	62	55	73	72	58
At 95° F.	139	160	153	157	146	137	146	139	121	157	150	109
Ductility, 77° F., cm	28	61	111	23	15	14	22	21	18	66	66	67
Rebound, percent	63	46		48	40	42	13	20	27	66	73	64
Residue from thin-film oven test:												
Penetration, 77° F.	65	57	49	47	43	40	47	47	43	57	54	49
Softening point, °F.	129.0	134.5	140.0	137.8	141.0	145.0	140.3	140.5	145.5	151.0	154.0	158.0
Ductility, 77° F., cm	+250	+250	45.0	10.0	9.5	9.0	11.8	11.0	10.5	48	43	41.5
Rebound, percent			29	52	47	50	43	42	48	59	63	71

in order to apply or mix the various rubber-asphalt blends under the same viscosity conditions as for the unblended asphalt AC–1, it would be necessary to raise the mixing temperature approximately 20° F. for the R–2 blend and 90° F. for the R–13 blend.

Effect of Mixing Temperature

The previous data given in this report has been for blends of asphalt and rubber prepared at 300° F. It seemed of importance to get some information concerning the effect of mixing at other temperatures on the properties of the rubber-asphalt blends. For this study asphalt AC–1 and rubbers R–1, R–2, R–6, and R–13 were used. Mixing temperatures of 300°, 338°, 375°, and 425° F. were employed to obtain the information desired.

Since rubbers R–1 and R–13 had been found to produce wide changes in the properties of asphalt, the mixing time for these was limited to 2 hours at 300°, 338°, and 425° F. Rubbers R–2 and R–6, which showed little change in the properties of the asphalts when mixed for 2 hours at 300° F., were mixed for 4 hours at 300°, 375°, and 425° F. Softening-point tests were made during the mixing periods, and the blends after being mixed for 2- or 4-hour periods were tested for penetration, ductility, and effect of heating in thin films at 325° F. The results of tests showing the effect of mixing temperature on the rubber-asphalt blends are given in table 14. The effect of heating at various temperatures on the softening point during the mixing period is shown in figure 18.

The blend containing rubber R–1 is of particular interest. When heated at 300° F. the softening point increased during mixing over the full 2-hour heating period, but when mixed at 338° and 425° F. there was a gradual decrease in softening point after the first 30 minutes. This reversal is apparently due to the instability of the natural rubber when subjected to the higher temperatures. The blend containing rubber R–13 showed an increase in softening point during mixing at the three temperatures with the rate of increase being greater with increases in mixing temperature. At all temperatures, blends containing rubbers R–2 and R–6 showed a relatively low rate of change in softening point during the mixing period.

Figure 19 shows the effect of mixing temperature on the penetration, softening point, and ductility of the blends after 2- and 4-hour periods. Except for rubber R–1, the penetration of the blends decreased as the temperature of mixing was increased. The blend containing the natural rubber R–1 had a higher penetration when mixed at 338° F. than at 300° F., and then decreased at 425° F.

Except for rubber R–1, the softening points of the blends were increased by raising the mixing temperature. As previously stated, the decrease in softening point for the blends made with the natural rubber R–1 shows the instability of this rubber when subjected to high temperatures. The greatest increase in softening point occurred with rubber R–13.

The ductility of the blends containing rubbers R–2, R–6, and R–13 was affected only slightly by increasing the temperature of mixing. The ductility of the blend containing rubber R–1 was increased greatly by mixing at the higher temperatures.

Effect of Rubber on Various Penetration Grades of the Same Asphalt

All of the rubber- asphalt blends which have been discussed previously in this report were prepared with 85–100 penetration asphalts. In order to get some information on the effect of rubber on other grades of asphalt cement, blends were prepared using 50–60, 85–100 and 120–150 penetration asphalts, refined from Venezuelan crude by the same producer. The 85–100 penetration grade is the same as the asphalt designated in this article as AC–1. Blends of these three asphalts containing 5 percent of the natural rubber R–1 were made by heating at 300° F. for 2 hours. Softening-point tests were made on samples taken during the mixing period and the resulting blends tested as in the previous studies. The results

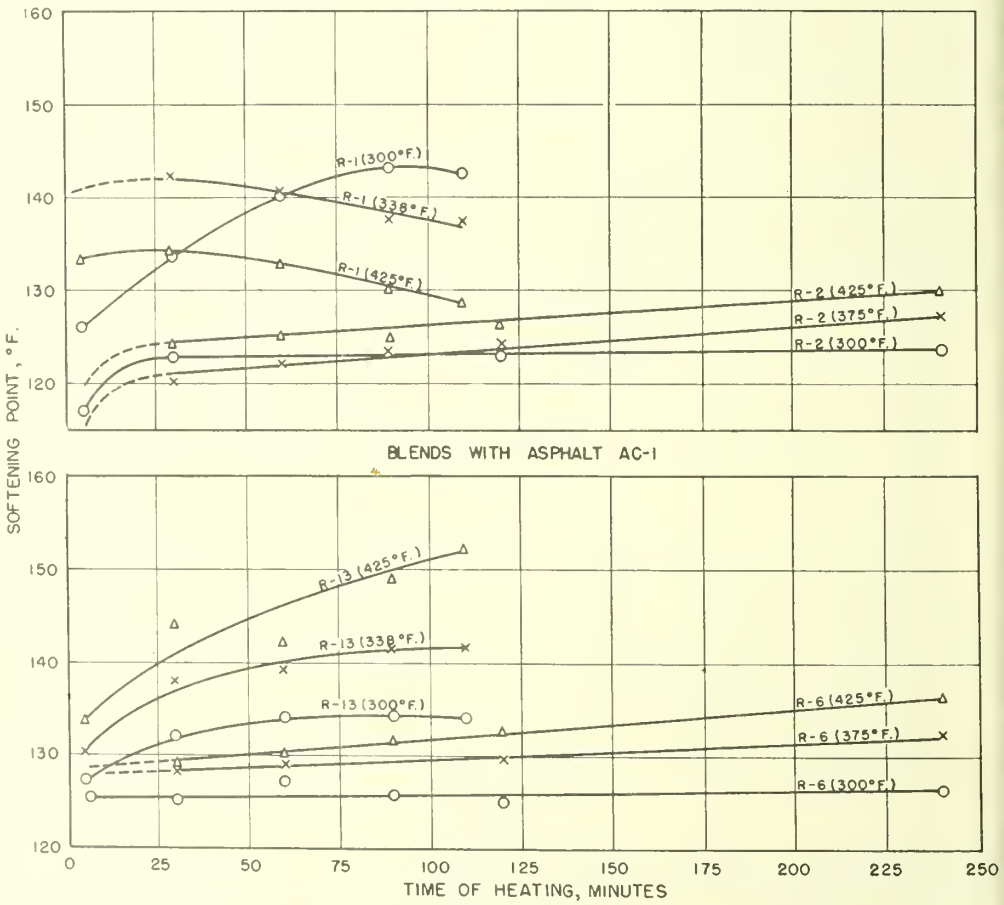

Figure 18.—Effect of temperature and time of heating on the softening point of rubber-asphalt blends.

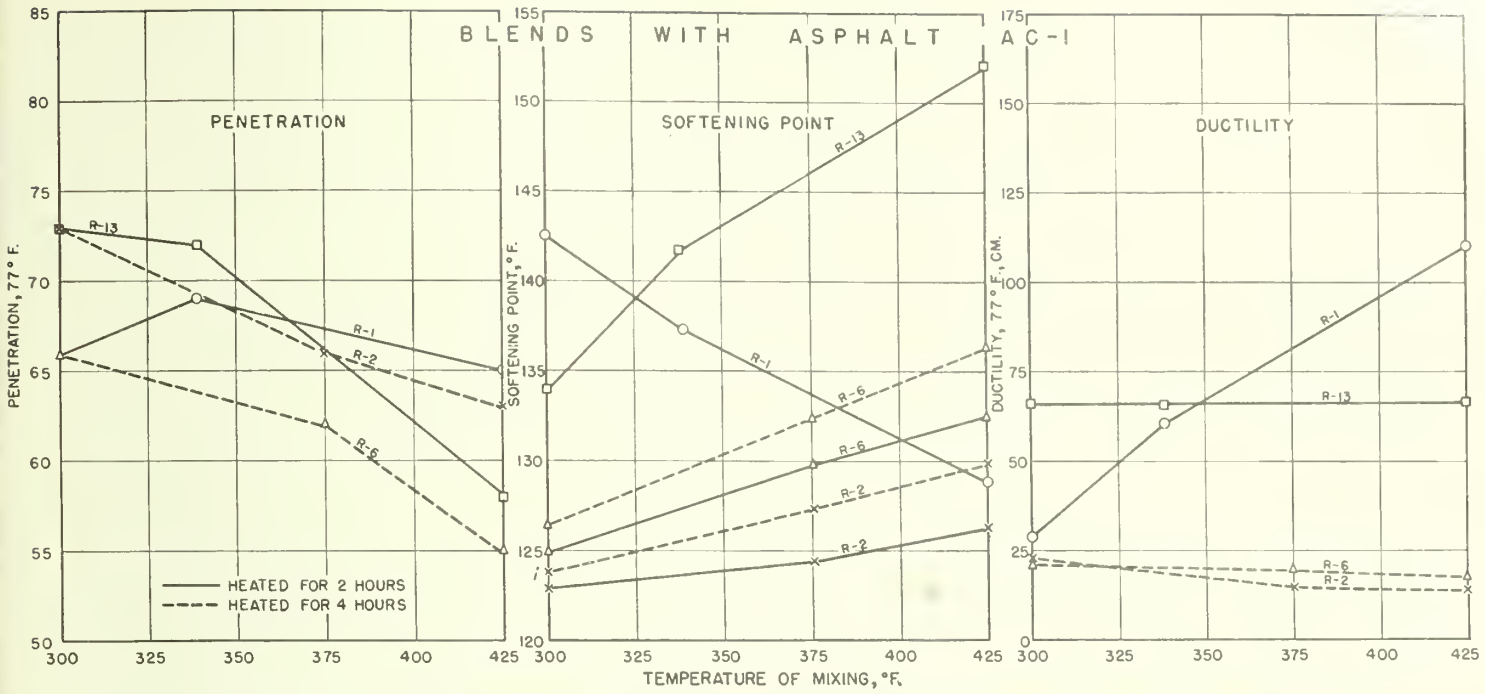

Figure 19.—Effect of mixing temperature on penetration, softening point, and ductility of rubber-asphalt blends.

of tests on the unblended asphalt and the rubber-asphalt blends are given in table 15.

Figure 20 shows the changes in softening point during the 2-hour mixing period for the blends prepared with the three grades of asphalt. The curves are of the same general shape and the difference in softening point remained fairly constant during the heating period.

In figure 21 the penetration, softening point, and ductility of the unblended asphalts are plotted against the penetration, softening point, and ductility of the corresponding rubber-asphalt blends. This figure indicates that as the penetration of the unblended asphalt was increased, the blends increased in penetration and had lower softening points and higher ductilities.

An analysis of the changes that were caused by the addition of rubber to the three grades of asphalt can be made from figure 22 where the penetration-softening point relations are plotted for the unblended asphalts and the rubber-asphalt blends before and after the thin-film oven test. For each of the above conditions the points for the three grades of asphalt are approximately linear and the curves thus formed are nearly parallel. The relative position of the curves for the original asphalts and for the rubber-asphalt blends shows the high increase in softening point with a corresponding decrease in penetration. After the thin-film oven test, the curve for the rubber asphalt blends shows a large decrease in softening point accompanied by a slight increase in penetration. The position of this curve is only slightly above the curve for the thin-film oven residues on the unblended asphalts. This again demonstrates the variable properties that rubber-asphalt blends of this type might have when subjected to heat and oxidation in the mixing and laying of hot asphalt mixtures.

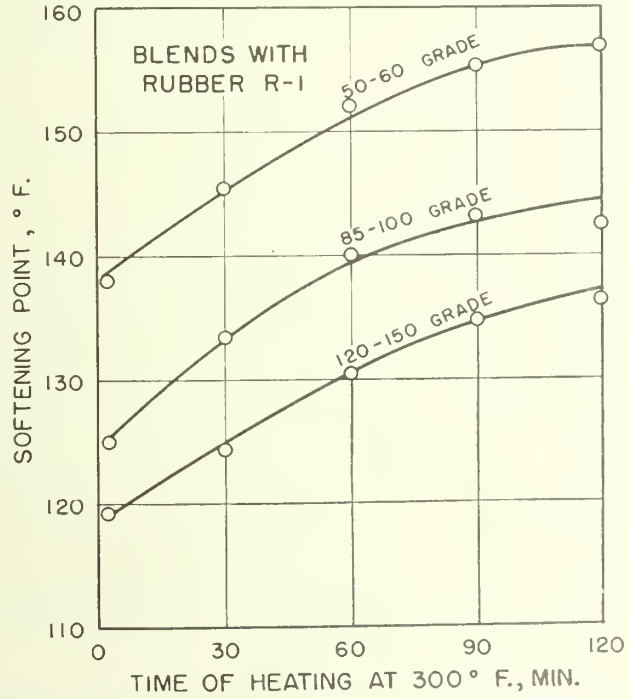

Figure 20.—Effect of time of heating on blends of 5 percent rubber R-1 with various grades of asphalt from the same source.

Recovery Tests on Mixtures

Tests on bitumens extracted and recovered from asphaltic paving mixtures have been useful in studying the changes that the contained asphaltic materials undergo during plant mixing with hot aggregate and during the exposure of bituminous pavements to service conditions. Usually these tests are made by extracting the soluble portion of the mixture with benzene and recovering this material by a method of test developed by Abson or by a modification thereof. Investigations have shown that asphalts are hardened during the hot mixing process or by exposure, and the amount of this hardening varies with the asphalt used and the conditions, such as temperature and the time of mixing or the climatic conditions, to which the mixture is subjected.

Tests on Recovered Bitumen

Since it has been shown that many of the rubbers included in this study are relatively insoluble in benzene, it was of interest to determine the effect of extraction and recovery tests on bitumens from asphalt mixtures in which rubber was used. Samples were obtained of the original asphalts and of paving

Table 15.—Effect of 5 percent natural rubber on asphalts of different penetration grades from the same source when mixed at 300° F. for 2 hours

	Unblended asphalt of penetration grade—			Blend of 5 percent R–1 with asphalt of penetration grade—		
	50–60	85–100	120–150	50–60	85–100	120–150
Softening point after mixing—						
5 minutes, °F	131.0	118.0	109.0	138.0	125.0	119.3
30 minutes, do				145.5	133.5	124.3
60 minutes, do				152.0	140.0	130.5
90 minutes, do				155.5	143.0	134.8
120 minutes, do				157.0	142.5	136.3
Tests on material after mixing for 2 hours:						
Penetration—						
At 50° F		20		13	22	22
At 77° F	50	89	136	39	66	85
At 95° F	122	232		85	139	195
Ductility, 77° F, cm	230	+250	152	17	28	40
Rebound, percent				65	63	79
Thin-film oven test:						
Loss, do	0.18	0.28	0.36	0.19	0.19	0.71
Tests on residue:						
Penetration, 77° F	33	51	74	43	65	99
Softening point, °F	143	131	123	138.8	129.0	118.8
Ductility, 77° F, cm	63	237	207	80	+250	240

mixtures in which these asphalts were used with and without rubber from several of the experimental pavements laid in the United States. These paving mixtures contained natural, reclaimed, synthetic, and plasticized rubbers and the powdered rubbers were the same as, or similar to, the rubbers R–1, R–2, R–5, and R–6 used in this study.

Extraction tests were made on these paving mixtures, using benzene, and the soluble material was recovered by the Abson method. The recovered bitumens were tested for penetration, softening point, and ductility. The results of tests on the original asphalts and on the bitumens recovered from the paving mixtures without rubber and corresponding mixtures which contained rubber are given in table 16.

Except for the mixtures containing the plasticized rubber, it was noticed that the extracted aggregates contained particles of rubber which had not been dissolved by the asphalt in the mixture or by the benzene used in the extraction test. Since it has been shown that the solubility of the rubber powders and rubber-asphalt blends in benzene are not the same, it was not possible to determine the exact amount of rubber actually contained in the bitumens recovered from the asphaltic concretes. It has also been shown that a wide range in test characteristics can be obtained with the natural rubber-asphalt blends, depending upon the asphalt used and the time and temperature used in the blending or mixing.

Comparisons of the characteristics of the original asphalts with those of the same asphalts recovered from the paving mixtures show that all the asphalts were hardened during the mixing process as evidenced by decreased penetration and increased softening point. Also, the ductilities of the recovered asphalts were lower than those of the original asphalts.

Comparisons of the characteristics of the bitumens recovered from the mixtures containing rubber with those of the bitumens recovered from the mixtures without rubber show that, in many cases, the presence of rubber increased the penetration; that in some cases it lowered the softening point; and that in most cases it increased the ductility. This apparent effect of rubber is contrary to its effect in the blends of rubber and asphalt prepared in the laboratory where, as compared with the unblended asphalts, the blends generally had lower penetration, higher softening point, and lower ductility.

These differences between the characteristics of rubber-asphalt blends prepared in the laboratory and the characteristics of bitumens extracted and recovered from paving mixtures containing powdered rubber are attributed partly to the fact that the mixtures of rubber powder and asphalt in the pavement are not directly comparable to the laboratory blends and partly to the fact that the bitumen recovered from the pavement is not the same as the mixture of rubber powder and asphalt as it exists in the pavement.

In the District of Columbia experiment two types of paving mixtures were laid (*23*). Except where the plasticized rubber was used, 60–70 penetration asphalt was used in the sand asphalt mixtures. Except for the control section, 85–100 penetration asphalt was used in the sheet-asphalt mixtures. The 60–70 penetration asphalt was used in the control sections for both sand-asphalt and sheet-asphalt mixtures and 85–100 penetration asphalt was used in the plasticized rubber sections for both types of mixtures. The amount of each kind of rubber used was the same in both types of mixtures.

Except where the plasticized rubber was used, all of the rubbers were incorporated into

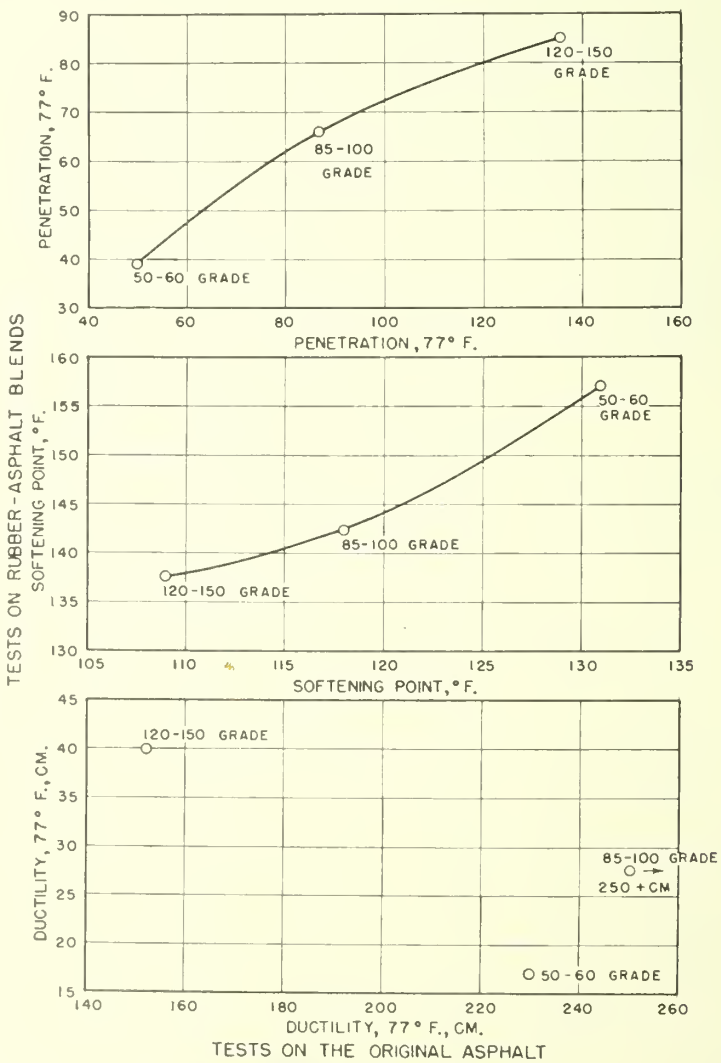

Figure 21.—Relation between tests on various grades of asphalt from the same source and tests on the same asphalt containing 5 percent rubber R–1.

Table 16.—Results of tests on asphalts and rubber-asphalt blends before and after recovery from paving mixtures

Type of mixture	Original material			Asphalt recovered from paving mixture		
	Penetration, 77° F.	Softening point	Ductility, 77° F.	Penetration, 77° F.	Softening point	Ductility, 77° F.
		°F.	Cm.		°F.	Cm.
Non-skid mixture (Ohio):						
Plain asphalt	70	122.0	175	47	132.4	136
Asphalt with natural rubber	------	------	-- --	43	135.5	78
Asphaltic concrete (Ohio):						
Plain asphalt	75	122.7	170	45	135.0	98
Asphalt with natural rubber	------	-- -	------	46	135.9	103
Asphalt with reclaimed rubber A	------	- - -	------	64	131.9	100
Asphalt with reclaimed rubber B	- - ---	--- -	-----	64	131.9	99
Asphalt with synthetic rubber	- ----	- - -	-	44	137.0	65
Asphaltic concrete (Texas):						
Plain asphalt	94	120.7	164	59	129.9	130
Asphalt with natural rubber	------	------	------	66	128.5	134
Asphalt with reclaimed rubber	------	------	--	62	129.9	175
Asphaltic concrete (Virginia):						
Plain asphalt	89	114.4	214	74	122.7	163
Asphalt with natural rubber	------	---- -	------	89	118.0	250
Sand-asphalt mixture (D. C.):						
Plain asphalt, 60–70 penetration [1]	64	123.0	+250	58	126.0	187
Asphalt with natural rubber [2]	45	141.5	32	43	131.9	+250
Asphalt with synthetic rubber [2]	53	135.0	25	61	126.3	+250
Asphalt with reclaimed rubber [2]	47	133.8	18	53	128.0	+250
Asphalt with plasticized rubber [3]	78	126.4	34	63	132.0	188
Sheet-asphalt mixture (D. C.):						
Plain asphalt, 60–70 penetration [1]	64	123.0	+250	53	128.6	225
Plain asphalt, 85–100 penetration [4]	93	117.0	+250	---	-----	-----
Asphalt with natural rubber [2]	63	134.0	33	65	125.8	+250
Asphalt with synthetic rubber [2]	70	129.1	22.5	88	118.5	+250
Asphalt with reclaimed rubber [2]	66	126.3	16	78	120.8	222
Asphalt with plasticized rubber [3]	78	126.4	34	61	133.5	198

[1] Used in all sections of sand asphalt, except in plasticized rubber blends, and in control sections for sheet asphalt.
[2] Rubber-asphalt blend prepared in laboratory by mixing at 300° F. for 2 hours.
[3] Plasticized rubber blended with asphalt at refinery and added to aggregate at mixing plant.
[4] Used in all rubber-asphalt blends for sheet asphalt and in plasticized rubber blend for sand asphalt.

the mixtures by adding them to the aggregate prior to the asphalt. The plasticized rubber was blended with asphalt at the refinery. Therefore, the characteristics of the powdered rubber-asphalt blends were not known as they existed in the asphalt concrete. In order that some comparison could be made between the powdered rubber-asphalt blends before and after recovery, laboratory blends were prepared with the rubber powders and asphalt cements used in the District of Columbia experiment in the same proportions as in the paving mixtures. These blends were made by mixing the asphalt and rubber for 2 hours at 300° F. The test characteristics of these materials, together with those for the plasticized rubber blend, are given in table 17. Figure 23 shows the effect of the time of heating on the softening point of the blends of the 60–70 and 85–100 penetration asphalts with the powdered rubbers. The results of the penetration, softening point, and ductility tests on the laboratory-prepared rubber-asphalt blends also are shown in table 16 for comparison with the same tests on the recovered bitumens.

A comparison of the laboratory-prepared rubber-asphalt blends with the bitumens recovered from the paving mixtures shows that, in general, the recovered bitumens had higher penetrations, lower softening points and very much higher ductilities than did the laboratory-prepared blends. It is recognized that the blends prepared in the laboratory are not necessarily comparable with the combinations of rubber powders and asphalts as they existed in the pavement. If it can be assumed that they had somewhat similar characteristics, the differences in penetration, softening point, and ductility which have been noted could be accounted for, at least partially, by the fact that free particles of rubber are lost from the blend during extraction.

In the case of the plasticized rubber blend there is opportunity for a direct comparison between the original blend and the blends extracted from the paving mixture. In this case, in contrast to the powdered rubber blends, the recovered bitumen had a lower penetration and a higher softening point than did the original blend. However, as in the case of the laboratory blends, the ductility of the recovered bitumen was much higher than the ductility of the original blend.

From the data given in tables 16 and 17 on the extraction and recovery tests, on the laboratory-prepared blends, and on the plasticized rubber blend prepared at the refinery, it can be seen that there was little or no correlation between the properties of rubber-asphalt blends and the properties of the blends recovered from the asphaltic paving mixtures, as measured by the penetration, softening point, and ductility tests.

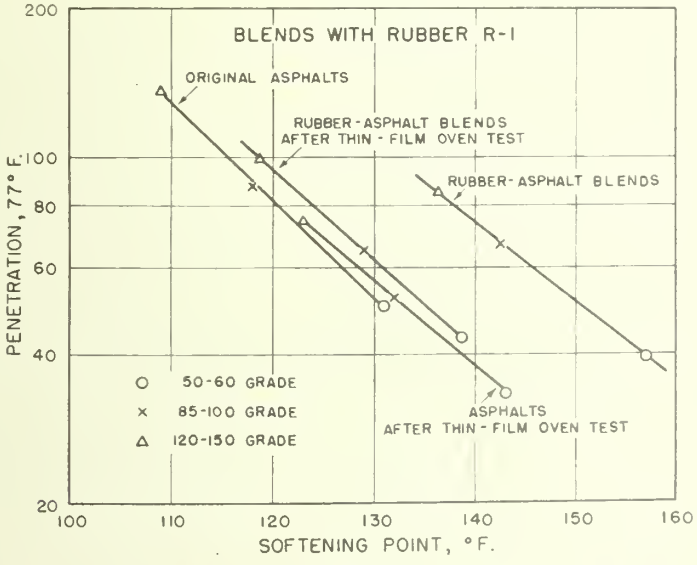

Figure 22.—Relation between softening point and penetration of various grades of asphalt from the same source with and without 5 percent rubber R-1, before and after thin-film oven test.

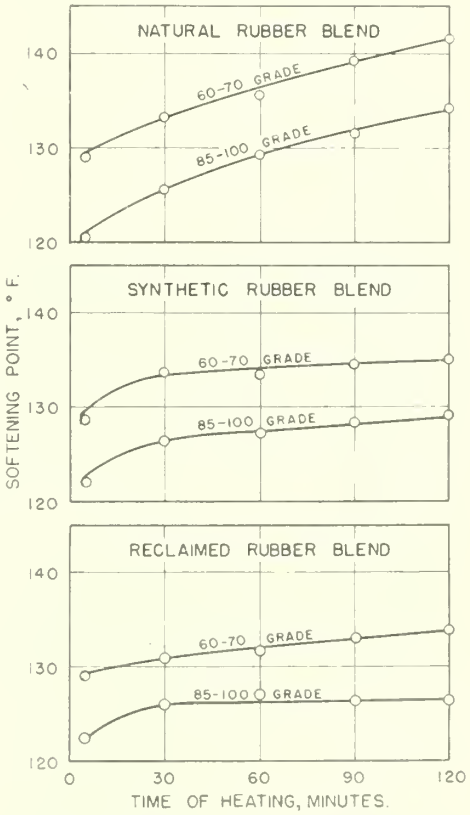

Figure 23.—Effect of time of heating on softening point of blends of various rubbers with 60–70 and 85–100 penetration grades of asphalt (used in District of Columbia experimental pavement).

Table 17.—Results of tests on laboratory blends of asphalt and various rubbers used on District of Columbia experimental pavements

	60–70 penetration asphalt blended with rubber indicated				85–100 penetration asphalt blended with rubber indicated				
	None (unblended)	5.2 percent natural	5.2 percent synthetic	7.0 percent reclaimed	None (unblended)	5.2 percent natural	5.2 percent synthetic	7.0 percent reclaimed	25.0 percent plasticized [1]
Softening point after mixing:									
5 min.....°F.	123.0	129.0	128.6	129.0	117.0	120.3	122.0	122.5	126.4
30 min.....°F.	-	133.0	133.6	130.8	-----	125.3	126.5	126.0	-----
60 min.....°F.	-----	135.5	133.5	131.5	-----	129.2	127.0	127.0	-----
90 min.....°F.	-----	139.0	134.5	132.8	-----	131.5	128.5	126.3	-----
120 min.....°F.	-----	141.5	135.0	133.8	-----	134.0	129.1	126.3	-----
Tests on mixture after heating 2 hours:									
Penetration, 100 g., 5 sec.:									
At 50° F.	13	12	16	11	18	18	16	16	21
At 77° F.	64	45	53	47	93	63	70	66	78
At 95° F.	192	105	120	121	275	137	174	162	190
Ductility, 77° F.....cm.	+250	32	25	18	+250	33	22.5	16	34
Rebound.....percent.	-----	78	60	31	-----	58	50	27	32
Thin-film oven test:									
Loss, 5 hr., 325° F.....percent.	0.15	0.14	0.14	0.19	0.27	0.24	0.24	0.27	0.39
Tests on residue:									
Penetration, 77° F.	38	54	36	33	54	69	48	45	50
Softening point.....°F.	132.8	128.0	146.0	148.5	126.5	124.5	141.0	142.0	139.0
Ductility.....cm.	+250	+250	30.5	14.5	188	+250	42	17.0	24.5
Rebound.....percent.	-	-----	66	62	----	-----	70	64	27

[1] Tests made on rubber-asphalt blend prepared at refinery with 85–100 penetration asphalt.

Table 18.—Viscosities of blends of SC–3 liquid asphalt and 5 percent of various rubber powders after periods of aging at room temperature

	Viscosity [1] at 95° F. after—						
	1 day	3 days	7 days	16 days	56 days	127 days	400 days
	Poises	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*	*Poises*
Unblended SC–3	160	157	155	162	155	155	155
Blend of SC–3 and—							
R–1	285	335	390	383	443	555	425
R–2	210	228	230	230	240	240	235
R–6	310	313	310	350	335	330	280
R–10	605	550	580	605	675	750	665
R–13	350	610	780	1,360	1,500	1,510	1,530

[1] Determined by Brookfield "Synchro-Lectric" viscometer with spindle No. 6 and speed of rotation of 2 r.p.m.

Therefore, to obtain at this time a definite idea as to the effect of rubber powders on the physical properties of asphalt, recourse must be made to tests of rubber-asphalt blends prepared in the laboratory. It should be noted, as indicated in table 16 and as has been shown by other investigations, that the properties of normal asphalt cements may be considerably altered during the mixing and laying operation. If similar changes take place in the characteristics of the rubber-asphalt blends during the mixing and laying operation, they cannot be determined at present because no extraction and recovery method available can recover the rubber-asphalt blend as it exists in the pavement.

Effect of Rubber on Liquid Asphalt

This report so far has been confined to the effect of rubber on asphalt cements when combined at relatively high temperatures. It has been demonstrated that variations in these high mixing temperatures produced changes in the characteristics of some of the rubber-asphalt blends. In order to determine the effect of the various rubbers on asphaltic materials when mixed and stored at normal atmospheric temperatures, a separate study was made with a nonvolatile, slow-curing, liquid asphaltic material of the SC–3 grade. The test characteristics of this material were as follows:

Specific gravity, 77°/77° F.	0.987
Saybolt Furol viscosity, 140° F.	511 sec.
Distillation test:	
Volume of distillate to 680° F.	0.0 percent
Tests on distillation residue:	
Float test of 122° F.	88 sec.
Residue of 100 penetration	79.4 percent
Tests on residue:	
Penetration, 77° F.	102
Ductility, 77° F.	88 cm.

Blends were prepared at room temperature with this material and 5 percent of rubber powders R–1, R–2, R–6, R–10, and R–13. After thorough mixing, the blends were sealed in 1-quart friction-top cans and stored in the laboratory. After periods of 1, 3, 7, 16, 56, 127, and 400 days the blends were heated to 95° F., stirred to insure uniformity, and their viscosity determined at 95° F. by means of the Brookfield viscometer. The results of these tests on the original SC–3 liquid asphalt and the various rubber blends are given in table 18. The changes in the viscosity of these materials are shown in figure 24, where the time of storage is plotted against the viscosity.

There was a very large difference in the change in viscosity for the various rubber blends during storage. After 1-day storage, all the blends had higher viscosity values

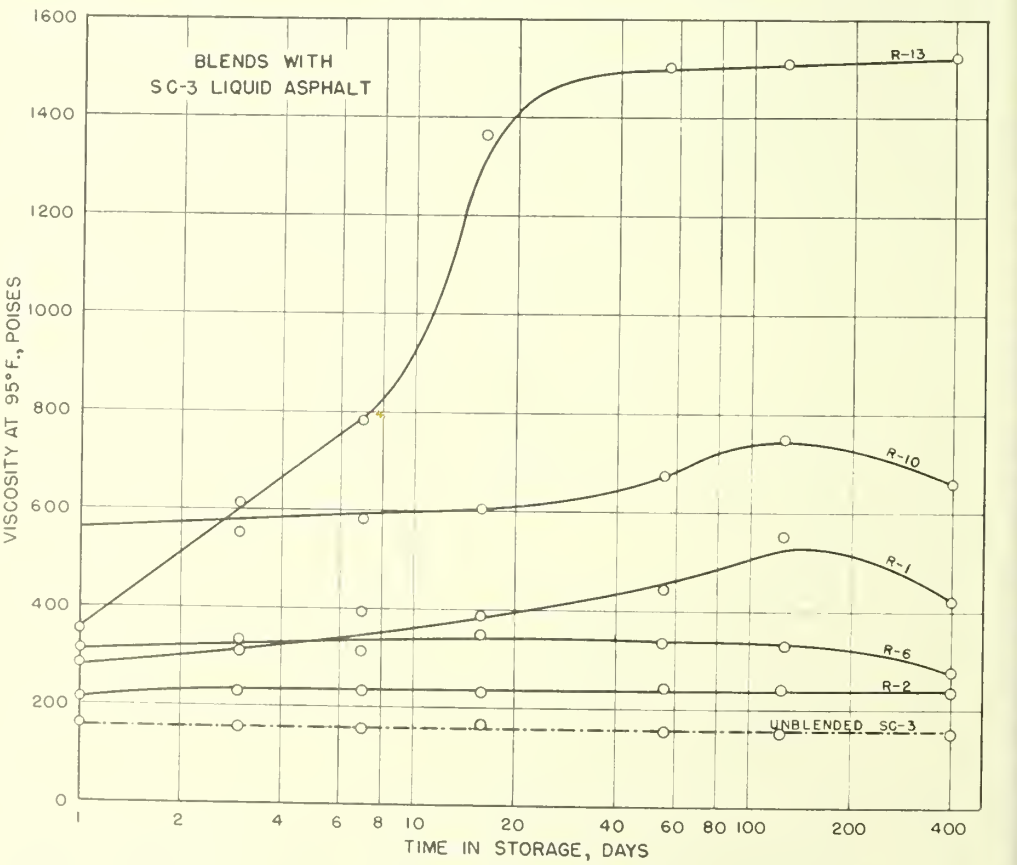

Figure 24.—Change in viscosity of blends of SC–3 liquid asphalt with 5 percent of various powdered rubbers during storage at room temperature.

Figure 25.—Effect of rubber on flow and elastic properties of SC–3 liquid asphalt: above, unblended asphalt; below, rubber-asphalt blend.

than the original SC–3 material. The range in viscosity was from 210 to 605 poises. Increasing the storage period to either 56 or 127 days increased the viscosities of all the blends, after which some of the blends showed a slight decrease in viscosity. At 16 days storage the range in viscosity was 230–1,360 poises and at 127 days 240–1,510 poises. The reclaimed rubber R–2 showed the least effect of the storage period and rubber R–13 the greatest effect.

The viscosity of the blend containing rubber R–1 increased gradually during storage up to 127 days but did not develop the high viscosity that would be expected when compared with the viscosity of the blends prepared with the same rubber and the asphalt cements. This may be attributed to the effect of temperatures at which the respective blends were prepared. The relative order of the viscosity developed in storage by the other blends of SC–3 asphalt and rubber is the same as for the blends of asphalt cements with the same rubbers. Figure 25 shows the effect of the GR–S Type II rubber, R–13, on the flow and elastic properties of the SC–3 material.

Theories about Rubber–Asphalt

During the development of the use of rubber powder as a modifier for asphaltic materials, several theories have been advanced as to what happens when these two materials are combined. In a report by van Der Bie and Kain (*1*) in 1938, the following theory was advanced: Asphaltic bitumens are regarded as a colloidal system of particles, rich in asphaltenes, in a medium of malthenes. When rubber is added, the asphaltenes remain unchanged while a portion of the rubber is dissolved by the malthenes, thus increasing their viscosity and, consequently, that of the entire system. Initially the greater part of the rubber remains in the form of granules and absorbs a portion of the malthenes which cause them to swell. This causes the remaining bitumen phase to become harder.

In another report, van Rooijen (*1*) in 1941 gave the following explanation of the effect of natural rubber on asphaltic bitumens: The addition of rubber powder to asphaltic bitumen creates a disperse system in which the rubber particles, which have swollen, are dispersed in the bituminous medium. When the swelling of the rubber is sufficiently great, the volumes of the two phases may be about equal. A reversal of phases may then occur in which the rubber becomes the continuous phase. This may be accompanied by a coagulation of the asphaltenes by the depolymerized rubber. This theory was used to explain why rubber-asphalt mixtures first become softer on heating to high temperatures and with continued heating again become harder.

Coltof (*24*) in 1937 studied the solubility properties of certain highly polymeric substances, paying special attention to the phenomenon of dissolution in regard to dispersion, swelling, and ballast action. He observed that the primary phenomenon is swelling. The swollen material grows and changes into a gelatinous mass and ultimately becomes completely miscible with the solvent. Coltof stated that there are two kinds of interaction between a liquid (simple or mixture) and a highly polymeric substance: Miscibility in all proportions (resulting in a solution or in a jelly) according to the fluidity of the mixture, and limited swelling (part of the liquid being absorbed by the particles while a certain part of the substance passes into the remainder of the liquid). He states further that heating tends to increase dispersion and decrease swelling.

De Decker (*25*) in 1951 stated that the behavior of dispersed rubber powders which causes the changes in the physical properties of rubber-asphalt blends is not yet fully explained and the research in the behavior of the rubber-asphalt binding material in a complete asphalt mixture is only in the first stage.

Photomicrographs of Blends

During the preparation of the rubber-asphalt blends, it was evident that many of the rubber powders used in this study retained their particle form even when subjected to very severe heat treatment, while blends containing other rubber powders became homogeneous under less severe conditions of mixing. A microscopic study of materials of this nature is dependent upon the ability to prepare films sufficiently thin to transmit light. It was found that for some of the rubber powders, which were of relatively large particle size and did not break down in the presence of asphaltic materials during mixing and heating, suitable films of the blends could not be prepared. In preparing the slides, a small drop of the rubber-asphalt blend was placed on a glass microscope slide, a cover glass was placed on

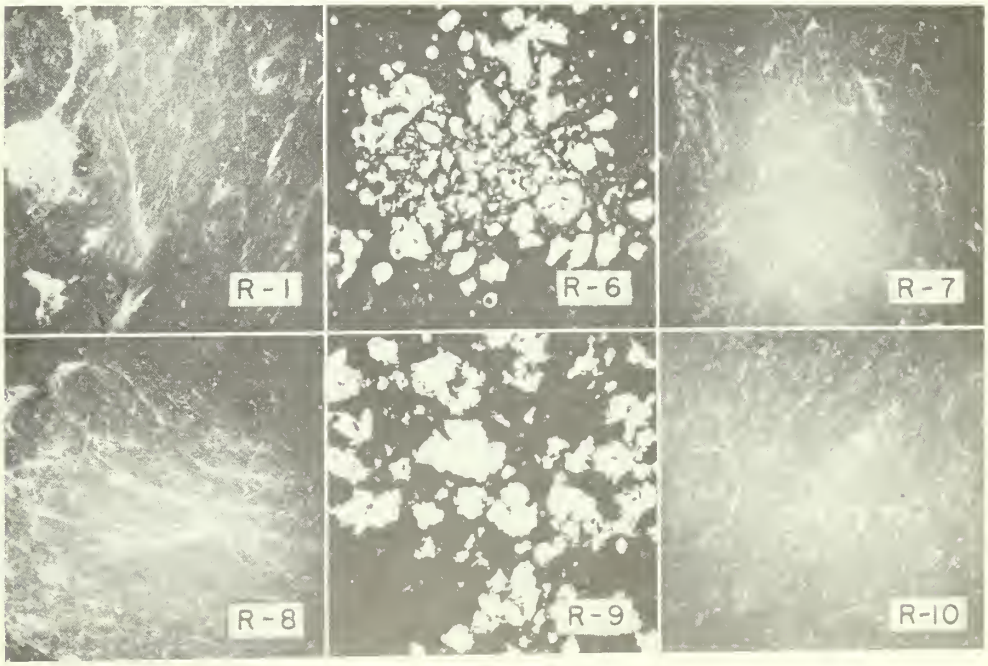

Figure 26.—Photomicrographs of blends of asphalt AC–1 with various powdered rubbers after heating and mixing at 300° F. for 2 hours (×34).

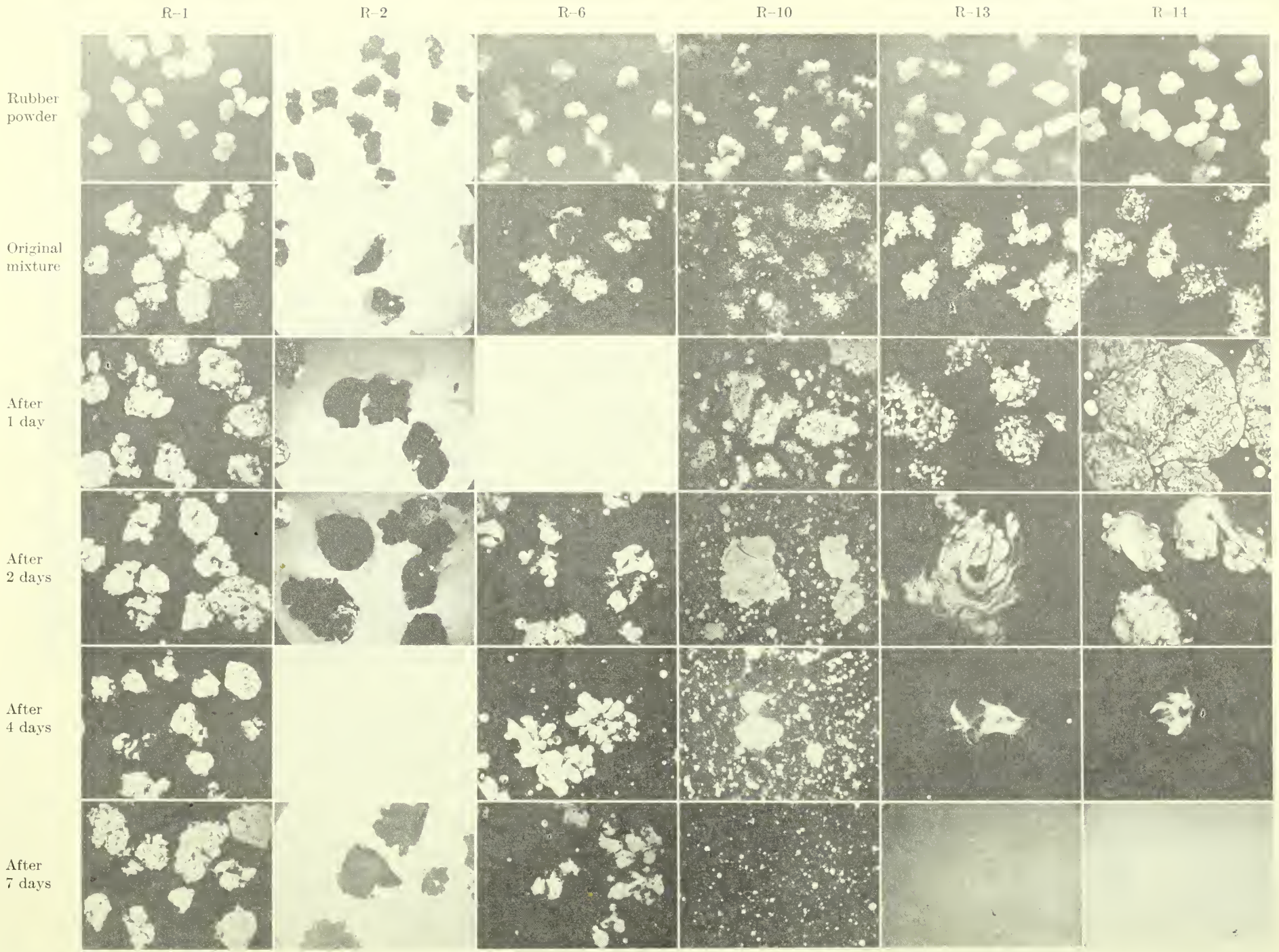

Figure 27.—Photomicrographs of various powdered rubbers before and after mixing with SC–3 liquid asphalt (×17½).
(No photomicrographs were available for R–2 after 4 days and R–6 after 1 day.)

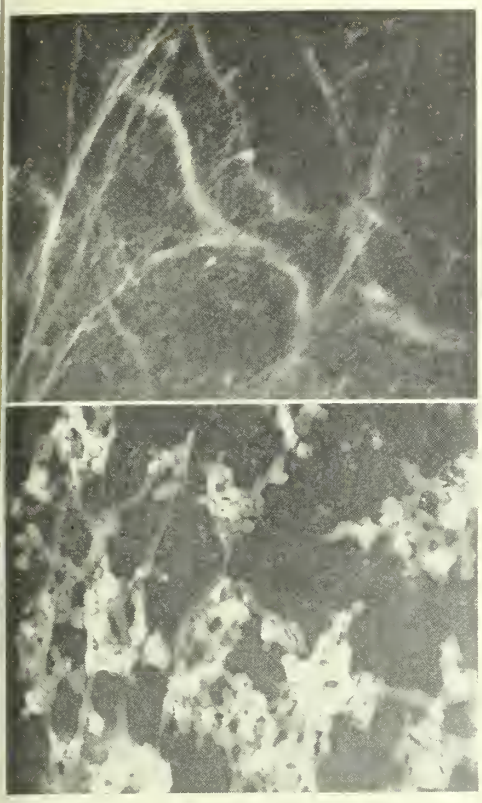

Figure 28.—Photomicrographs of blend of rubber R–14 with SC–3 liquid asphalt (×33). Above: Blend after 8 days, after standing 1 day without stirring before slide was made. Below: The same slide, 24 hours later, showing development of rubber structure on standing undisturbed.

top, and the assembly was warmed sufficiently so that light pressure applied to the cover glass caused the material to flow into a thin film. As mentioned above, where large particles of rubber were present between the cover glass and slide it was not possible to make the film sufficiently thin to transmit light.

Where satisfactory slides could be made, photomicrographs were taken of some of the rubber-asphalt blends prepared by heating for 2 hours at 300° F. The state in which the rubber existed in the asphalt at this time is illustrated by figure 26. Although photomicrographs of some of the rubber-asphalt blends are not shown here, the various rubbers appeared to fall into two groups depending upon their behavior. The natural rubber R–1 and the synthetic rubbers R–7, R–8, R–10, R–13, and R–14 appeared to have softened and were dispersed throughout the asphalt. The reclaimed, processed, and ground scrap rubbers R–2, R–3, R–4, R–5, R–15, and R–16 and the synthetic rubbers R–6 and R–9, except for some swelling, appeared to be relatively unaffected in the asphalt blend. It is of interest to point out the fact that those rubbers that showed the greatest change in particle structure when blended with the hot asphalt also produced the greatest changes in the properties of the rubber-asphalt blends as measured by the physical tests.

Detailed Photomicrographic Study

Since the first photomicrographic study showed a wide difference in appearance of the various rubbers when mixed with asphalt, a more detailed study was undertaken to determine the relative behavior of some of the rubber powders. To reduce the influence of the particle size, only that portion of powdered rubber passing a No. 50 sieve and retained on a No. 80 sieve was used. To eliminate the effect of heat, as used in preparing the blends with asphalt cements, the SC–3 liquid asphalt previously referred to was used so that blends could be made at room temperature. Blends were prepared with the SC–3 asphalt containing 5 percent of rubbers R–1, R–2, R–6, R–10, R–13, and R–14. Films of the blends were made immediately after mixing and after 1, 2, 4, and 7 days storage at room temperature. Photomicrographs were taken of these films and also of the rubber powder, as shown in figure 27.

During the 7-day period, the particles of rubbers R–1, R–2, and R–6 showed some swelling. There was some disintegration of individual particles of R–6 but none of R–1. The streaks shown in the blend with rubber R–2 at 1- and 2-day periods probably were due to some of the carbon black flowing from the rubber particles through the asphalt.

Rubber R–10 appeared to break into smaller particles during the initial mixing. After 1 day there was considerable swelling of the rubber, followed by a gradual breakdown and disappearance of the large particles. At 7 days, there were only a few small rubber particles visible.

The particles of rubbers R–13 and R–14 seemed to soften and swell very rapidly. After this condition the rubber appeared to be dissolved or dispersed in the asphalt, and at 7 days only slight traces of the rubber were visible.

The striking difference in the behavior of the natural rubber R–1 when mixed with asphalt cement (fig. 26) and with the SC–3 liquid asphalt (fig. 27) probably was due to the difference in mixing temperature, as previous work has shown this rubber to be very unstable when subjected to high temperatures.

As previously indicated, the changes in particle structure during the heating of the rubber powders could not be followed by photomicrographs, although it was apparent that those rubbers which formed homogeneous mixtures did swell to some extent, softened, and dissolved or were finely dispersed. The reclaimed and processed rubber and ground vulcanized scrap, as well as the GR–S Type V rubber R–6, did not swell to a great extent but did break down to smaller particles in the hot asphalt. When mixed with SC–3 liquid asphalt, natural rubber R–1, which formed a homogeneous mixture with the hot asphalts, swelled but slightly and retained its particle structure in the same manner as the more inert rubbers in the SC–3 liquid asphalt.

During the blending of rubbers R–13 and R–14 with the SC–3 liquid asphalt, it was noticed that these blends developed a high degree of gelation during storage and on stirring they became very rubbery. In an attempt to show this change in condition, slides of the blended rubber R–14 were prepared on the eighth day without stirring the mixture for 24 hours. Definite changes in the rubber structure were found to have occurred in these materials, as illustrated in figure 28, for the blend of SC–3 asphalt and rubber R–14.

The photomicrograph taken of the blend without stirring shows considerable more rubber structure than does the photomicrograph of the same blend after 7 days, shown in figure 27. The photomicrograph of the slide after 24 hours (figure 28, bottom) shows the rubber to have separated or coagulated to a much greater extent. These conditions no doubt account for the physical changes occurring in the blends of rubbers R–13 and R–14 with the SC–3 liquid asphalt. Changes such as these with the SC–3 liquid asphalt have not been noticed with the blends prepared with asphalt cements.

References to various theories that have been advanced (*1*) as to what happens when rubber powders are blended with asphaltic materials indicate that some investigators believe that the rubbers are soluble only in the malthene portion of the asphalts. Considering the variable solubilities of the rubber powders in simple solvents, such as those normally used in the examination of bituminous materials, and considering the changes in particle size and structure of the rubber blends with the asphalt and SC–3 slow-curing liquid asphalt, as shown in the photomicrographs, it might well be assumed that these asphaltic materials act as a solvent mixture. According to Coltof (*24*), therefore, these rubber powders, depending upon the degree of vulcanization, upon the presence of nonrubber bodies, and upon the temperatures involved in effecting solution, might act in the same manner as other highly polymeric substances in simple or complex solvents. The amount of dissolution, the dispersion, and the swelling that occur when blending the various rubbers with asphaltic materials may be due to the action of the asphaltic material itself and not to the selective action of its malthene fraction.

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A Laboratory Study of Rubber-Asphalt Paving Mixtures

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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DURING recent years, the use of several types of rubber admixtures in bituminous roads has been promoted intensively. The principal advantages claimed for bituminous road surfaces containing rubber over conventional pavements are lower susceptibility to temperature change, being less likely to crack in cold weather and less likely to be displaced by traffic in hot weather, greater resistance to the action of water, greater resistance to abrasion by traffic, higher resistance to skidding, and longer life, with lower maintenance costs.

Considerable interest in this idea has been shown by many States and cities in the United States, and has resulted in the installation of test sections on a number of city streets and State highways for the purpose of determining the value of adding rubber under actual service conditions. The Bureau of Public Roads participated in observing the construction of test sections in several of the States, through the courtesy of the respective highway departments. Skid-resistance measurements are being made periodically on some of these test sections by the State highway departments. Also, it is hoped that laboratory testing of samples cut from the test roads from time to time will show the comparative degree of progressive alteration in other properties of the mixtures that occurs with age.

In the field experiments, several different forms of rubber have been used: rubber latex, rubber powders or crumbs, and so-called plasticized rubber. The rubber latex, as used in the United States, has been a synthetic product. The rubber powders used have consisted of natural and synthetic materials, and of processed reclaimed rubber. Two methods of incorporating the rubber powders into the final paving mixture have been used: adding the powder directly to the aggregate in the mixing box just before adding the bituminous material, and preblending the rubber with the bituminous material and then mixing the blend with the aggregate. The so-called plasticized rubber is a proprietary commercial compound which has been used widely as a joint- and crack-filler. While its exact composition is not known, this product is believed to consist essentially of synthetic rubber and bitumen, and for the purpose of this study has been considered as a preblend of rubber and bituminous material.

Decisions as to the value of additive rubber derived from the relative service behavior of test roads must necessarily await the passage of considerable time. Study of the literature indicates that to date most of the laboratory research on the subject has been devoted to work with blends of asphalt and rubber, and very little to investigations of asphalt-rubber-aggregate mixtures. In order, therefore, to determine to what extent the addition of rubber to bituminous mixtures would change certain laboratory test properties of these mixtures, and thereby to arrive at conclusions, of a tentative nature at least, with respect to the value of adding the rubber, this laboratory study was initiated.

It was realized when planning the study, that interest in the development of new types and forms of rubber for use in bituminous paving is continuing. Likewise, new methods of incorporating the rubber materials now in use experimentally may be developed, or perhaps it may be found advantageous to combine the rubbers with types of bituminous materials other than those presently employed for the purpose. Meanwhile, this study was designed with a view to evaluating the materials and methods of incorporation that have been most widely used in this country to date.

Conclusions

Bearing in mind the limited scope of this study with respect to the number and kinds of rubber materials, the single proportion of rubber, and the single grade and source of asphalt employed, the test results are believed to warrant the following comments.

1. When added in the form of powder, the incorporation of rubber lowers the compactibility of an asphaltic paving mixture. In this regard the results of the laboratory tests were consistent with results of density tests that have been made on samples taken from several experimental road surfaces shortly after their construction. As tested in the laboratory, mixtures containing such rubber powders, when compacted under standard conditions, have lower stability and higher susceptibility to temperature change than control mixes without rubber.

2. In most of the field experiments installed in this country to date, the rubber has been incorporated by adding it in powder form to the aggregate before mixing with asphalt. Results of the laboratory tests show that mixtures prepared by preblending the rubber with asphalt are much more compactible and stable than corresponding mixtures containing rubber that has been added in the form of powder. In fact, mixtures containing three of the four rubber materials tested in preblended form showed higher stabilities than the control mixture without rubber.

3. In the results of compression tests of mixtures containing rubber powders molded under varied pressures to produce equally low densities, and especially in the results of mixes containing preblended rubber and asphalt, it is possible to discern a degree of relation between the changes in stability, plasticity, and susceptibility to temperature change of the mixtures in this study and the changes in penetration, softening point, and susceptibility obtained in previous laboratory studies made of blends of asphalt and rubber. Such relation is entirely lacking in results of compression tests on mixtures containing rubber powders compacted under standard pressure.

4. Although the results show that the plastic properties of pavements containing preblended synthetic, natural, and plasticized rubber and asphalt would, in their early life at least, respond at a slower rate to temperature change than surfaces without rubber, the data hold no indication that pavement surfaces containing any of the rubber materials tested would be both substantially more plastic at lower temperatures and substantially less plastic at higher temperatures than surfaces without rubber. Mixtures containing plasticized rubber and preblended natural rubber were less plastic at 140° F. than the control mix, but they were also less plastic at 77° F. In general, mixtures containing synthetic and reclaimed rubber in either powder or preblend form were more plastic at 77° F. than the control mix, but they were also more plastic at 140° F. The exception to this generality was the mixture containing preblended synthetic rubber which, after oven exposure for 21 days, was slightly less plastic at 140° F. and slightly more plastic at 77° F. than the control mix.

5. The addition of rubber in either powder or preblend form did not increase the resistance of the mixture to the action of water.

6. Mixtures consisting of Ottawa sand and preblended asphalt and natural rubber were found to be more resistant to abrasion than the control mix without rubber, mixes con-

taining the other preblended rubbers tested, and mixes containing the same natural rubber tested in the form of powder.

Planning the Study

Two general lines of approach have been used in planning laboratory and field studies of asphalt mixtures containing rubber. In one approach, mixtures consisting of aggregate and combinations of rubber and asphalt have been compared directly with control mixtures containing asphalts, the viscous properties of which were equivalent to those for the rubber-asphalt combinations as predetermined by laboratory tests on the latter materials. For example, if, when a certain rubber was added to a certain asphalt in the 85–100 penetration range, it was found that the penetration of the resulting combination was between 60 and 70, then an asphalt in the 60–70 penetration range would be selected for the asphalt-aggregate control mixture.

This approach entails a number of practical difficulties. In the first place, it has been found that different types of rubber vary with respect to their solubility in asphalt, and thus with respect to their effect on consistency. A test program involving several types of rubber might, therefore, require several control mixtures. In any case, the degree of mutual solubility of rubber and asphalt actually attained in any given blend or combination is very difficult to determine. Thus it is very difficult to decide whether observable differences in physical characteristics are attributable to properties of a complex substance (mutually dissolved asphalt and rubber) or to properties of asphalt in which an essentially inert material is dispersed.

Again, one of the claims made for pavements containing rubber is that they are less prone to reflect cracks in underlying rigid pavements. If, in a field experiment laid over such a pavement, cracking is found in a control section containing 60–70 penetration asphalt and is not found in adjacent sections containing blends of rubber and 85–100 penetration asphalt, the question logically arises as to whether cracking would have occurred in the control section had 85–100 penetration asphalt been used there as well as in the rubber section.

The other approach, and the one used in planning this study, confines the scope to consideration of one question only: namely, what is the effect of adding a small quantity of rubber to a bituminous mixture consisting of a given aggregate and a given asphalt?

Table 1.—Results of sieve analysis of rubber powders

	R-1 Natural	R-2 Reclaimed	R-6 Synthetic
Percentage passing by weight:			
No. 10 sieve	100	99	100
No. 20 sieve	97	69	100
No. 30 sieve	83	33	98
No. 40 sieve	57	14	90
No. 50 sieve	31	4	83
No. 80 sieve	8	1	71
No. 100 sieve	3		61

By using the same grade of asphalt in the control mixture as is used in the mixtures to which the rubber is added, variables other than the addition of rubber are eliminated, thus permitting clear-cut and rational comparisons.

Laboratory Tests Used

The principal objectives of this study, together with the test methods selected for the attainment of each, were as follows:

To determine the effect of the manner in which the rubber is added to the asphalt mixture, two methods were employed. In one, the rubber powder was added to the hot aggregate in the mixer prior to the addition of the asphalt. In the other, the rubber was preblended prior to mixing with the aggregate, by heating and stirring the rubber and asphalt for a period of time.

To determine the effect of rubber on the temperature susceptibility (the rate at which the consistency of the asphalt changes with temperature) of the mixture, and to determine the effect of aging on this susceptibility, unconfined compression tests were made at two test temperatures, 77° and 140° F., after the specimens had been cured for 1 day in an oven at 140° F. and after they had been exposed to this temperature in the oven for 21 days.

To determine the effect of rubber on the resistance of the mixture to the action of water, the immersion-compression test (ASTM designation D 1075–49T) was made on specimens that had been cured for 1 day in an oven at 140° F. and on similar specimens after they had been aged in the oven at this temperature for 21 days. Static immersion tests were also made.

To determine the effect of rubber on resistance to abrasion, the modified California abrasion test was made.

Details of Test Procedures

Specimens used in the compression testing were cylinders 3 inches in diameter and 3 inches high. Two specimens were molded from each batch. Except for the size of specimens, size of batch, and molding pressures used, the mixing and molding procedures followed were the same as in ASTM designation D 1074–52T.

In the immersion-compression testing, 12 test cylinders were prepared and placed in an oven maintained at 140° F. They were removed from the oven 24 hours later, allowed to cool to a temperature of 77° F., and then measured for bulk specific gravity, using the water-displacement method. They were then divided into four sets of three specimens each on the basis of the specific gravity data, so that the average specific gravity of each of the four sets was approximately the same as the average for the twelve specimens.

Of the four sets of test specimens, set 1 was tested in compression at 77° F. in a dry condition. Set 2 was placed in water at 120° F. for 4 days, at the end of which time the three specimens in the set were measured again for bulk specific gravity and tested in compression at 77° F. Sets 3 and 4 were replaced in the 140° F. oven immediately after the original density determinations had been made and kept there for 20 additional days. At the expiration of this oven-exposure period the specimens of sets 3 and 4 were removed from the oven and treated in the same manner as just described for sets 1 and 2, respectively. The ratio resulting from dividing the average compressive strength of the specimens of set 2, after immersion, by the average compressive strength of the specimens of set 1, tested dry, was taken as a measure of the effect of water on the mixture after 1 day of oven exposure. The ratio of average compressive strengths of specimens comprising sets 4 and 3 were taken as the measure of the effect of water on the mixture after 20 additional days of oven exposure at 140° F. The effect of the simulated aging provided by the additional oven exposure was obtained by comparing the two ratios, which represent retained strength values.

In the compression tests to measure stability, plastic properties, and susceptibility to temperature change, one set of three test specimens molded as described above was brought to a temperature of 77° F. in an air bath and tested in unconfined compression. A second set of three specimens was brought to a temperature of 140° F. and tested while immersed in water maintained at that temperature. Both of these two sets of specimens had been exposed to warm air in a 140° F. oven for 24 hours before testing. In order to determine the effect of aging on the properties of stability and plasticity of the mixture, two companion sets of specimens were kept in a 140° F. oven for 20 additional days, at the expiration of which period they were tested as just described.

All compression testing was done at a vertical-deformation rate of 0.05 inch per minute per inch of specimen height. Load deformation curves were obtained by means of an electrically actuated automatic recorder attached to the hydraulic testing machine.

The basic or control mixture used in all of the stability, temperature susceptibility, and immersion-compression tests consisted of hydrophilic crushed granite, concrete sand, commercial limestone dust, and 85–100 penetration asphalt.[1] These materials were combined to form a mixture containing (by weight) 5.5 parts of asphalt to 100 parts of aggregate graded as follows:

Passing:	*Percent*
⅜-inch sieve	100
No. 4 sieve	82
No. 10 sieve	37
No. 40 sieve	14
No. 200 sieve	5

As has already been indicated, only one asphalt was used in this study. As has also been stated in this report, asphalts and rubbers alike appear to differ with respect to their mutual solubility, so that, had additional

[1] This material is the same as that designated as AC–2 in the report, *The effect of various rubbers on the properties of petroleum asphalts* by R. H. Lewis and J. Y. Welborn, appearing on pp. 64–89.

asphalts from other sources been included in the study, a wider range of results might have been obtained. The test characteristics of the asphalt used are as follows:

Specific gravity, 77°/77° F.	1.012
Penetration 100 g., 5 sec.—	
At 50° F.	24
At 77° F.	94
At 95° F.	238
Softening point, °F.	120.0
Ductility, 77°F., cm.	195
Standard oven test, 5 hr., 325° F.:	
Loss, percent	0.02
Tests on residue:	
Penetration, 77° F.	76
Softening point, °F.	123.8
Ductility, 77° F., cm.	160
Thin-film oven test, 5 hr., 325° F.:	
Loss, percent	+0.01
Tests on residue:	
Penetration, 77° F.	56
Softening point, °F.	134
Ductility, 77° F., cm.	52
Bitumen soluble in CS_2, percent	99.94
Organic insoluble, do	0.02
Inorganic insoluble, do	0.04
Oliensis spot test	Negative

In preparing the control mix for the static-immersion test, the aggregate, which consisted of quartzite (a hydrophilic material) passing the ⅜-inch sieve and retained on the No. 4 sieve, was washed and heated to 300° F. The asphalt, which was also heated to 300° F., was mixed with the hot aggregate by hand over a hot plate until the best possible coating was obtained. The proportions used for the control mixture were 95 parts of aggregate to 5 parts of the asphalt, by weight. The mixture was then placed in an oven at 140° F. for 24 hours, after which it was cooled to room temperature and divided into three approximately equal portions. One of these portions was immersed in distilled water for 24 hours at a temperature of 100°F. The other portions were similarly immersed in water maintained at temperatures of 120° and 140° F., respectively. Upon completion of the water-immersion period, each sample was examined visually and the extent of stripping estimated. This examination was made without disturbance or agitation of the sample, and was made while the sample was still immersed in water. The area of the aggregate remaining coated with bitumen at the end of the test was reported as a percentage of the total surface area of the aggregate.

The basic or control mixture used for the abrasion test consisted of 100 parts of standard Ottawa sand (No. 20–No. 30) and two parts of asphalt, by weight. Both the sand and the asphalt were heated to 325° F. before mixing in the laboratory mixer. After mixing, the batch was spread out in shallow pans to a depth of about one-half inch and allowed to cool to air temperature. After cooling, 30-grams of the mixture were placed in each of eighteen 2-ounce ointment tins. These tins were then placed in an oven at 325° F. to warm the mix for molding.

The tins and contents were placed in the oven one at a time at 3-minute intervals. Molding was started after 9 minutes and continued at the rate of one specimen each 3 minutes so that the preheating time for each 30-gram lot was uniformly 9 minutes. The test specimens were formed and tested in steel cups having an inside diameter of 2 inches and a depth of ⅜ inch. Each batch of exactly 30 grams of warm mixture was placed in one of the warm molds and compressed under a load of 1,000 pounds per square inch maintained for 1 minute. After molding, the specimens in their steel cups were allowed to cool to 77° F. prior to being placed in the 325° F. oven for weathering.

Weathering periods of ¼, 1, 2, 4, and 8 hours were used. Three specimens were tested without oven exposure and at the expiration of each weathering period. After removal from the oven, the weathered specimens together with their supporting cups, were cooled to room temperature, weighed, and brought to test temperature of 77° F. in an air bath. The unweathered specimens were similarly weighed and brought to test temperature.

The apparatus used in the abrasion test was one that conformed essentially to the California design. The apparatus controls the fall of lead shot at the rate of 2,000 grams of shot per minute through a distance of 1 meter onto the surface of the test specimen. During the test the specimen is rotated at the rate of 340 r. p. m. The temperature of the test is controlled by placing the machine in an air bath held at 77° F. The shot is stored in the air bath between tests. The abrasion loss is normally reported on the basis of loss per 1,000 grams of shot after a total of 4,000 grams of shot has been dropped. After the test is stopped, the specimen is weighed and the loss determined to hundredths of a gram. Additional details of the test procedure are given in a previous report.[2]

Types of Rubber Studied

Three of the rubber materials used in this study were in powder form, and the fourth was a meltable rubber compound or plasticized rubber. The powdered materials consisted of a natural rubber from Indonesia, a processed reclaimed rubber, and a synthetic rubber.[3] Sieve analyses of these powders are given in table 1.

Two different methods were used to incorporate the powders in the various test mixtures: adding them to the hot aggregate before adding the asphalt, and preblending them with the asphalt. In all of the test mixtures, regardless of the method of adding the rubbers, the proportion of rubber powder used was 5 percent of the asphalt, by weight.

In preparing the test mixtures containing the plasticized rubber, the rubber compound and the 85–100 penetration asphalt were preblended in the proportion of 25 and 75 percent, respectively. Although information regarding the exact rubber content of the plasticized material itself has not been made available by the manufacturer of this commercial product, it has been estimated that the amount of rubber hydrocarbons in the blend of asphalt and plasticized rubber compound combined in the foregoing proportions is approximately 5 percent by weight of the asphalt.

In preparing the preblends of rubber and asphalt, the temperature of the asphalt was held at 300° F. and the various rubbers were stirred with the asphalt at that temperature for the following time periods:

Natural rubber powder	1½ hours
Synthetic rubber powder	2 hours
Reclaimed rubber powder	2 hours
Plasticized rubber compound	1 hour

The blends were then allowed to cool, and thereafter were heated again to 300° F. just before mixing with the aggregate, as needed.

In preparing all of the test mixtures containing the rubber powders, the proper amounts of asphalt, rubber, and aggregate for each batch were weighed separately and then combined in the order of their admission to the laboratory mixer, as follows: aggregate (preheated to 325° F.), rubber powder, and asphalt (preheated to 300° F.).

Comparing Temperature Susceptibility of the Mixtures

In 1949 and 1950, experimental asphalt-rubber road sections were built by several State highway departments. A number of samples of the mixtures from some of these field experiments were brought to the Bureau's laboratory for testing. In the course of these investigations, molded specimens of the mixtures were tested for unconfined compressive strength at 77° and 120° F., with the thought in mind that comparison of the compressive strengths of a particular mixture at the two test temperatures would provide some measure of its susceptibility to temperature change. At the same time, load-deformation curves were obtained by means of an automatic recorder attached to the testing machine. Study of these curves was made with a view to relating the character of the curve with the degree of plasticity of the mixture, and by this means to develop additional information as to the effect of temperature change on the flow properties of the mixture.

The behavior of compacted bituminous mixtures when subjected to vertical load without lateral support may differ widely, depending upon how closely they resemble rigid, elastic materials at the one extreme or plastic, easily deformable materials at the other. When tested at low temperatures, and where deflections or deformations of small magnitude are involved, bituminous mixtures may assume quasi-elastic properties; at high temperatures the same mixtures may exhibit extremely plastic properties.

Speed of testing (rate of loading or rate of vertical displacement) and ratio of height to diameter of the test specimen are other procedural factors that influence greatly the load-deformation relation with respect to any

[2] *Studies of the hardening properties of asphaltic materials*, by J. T. Pauls and J. Y. Welborn. PUBLIC ROADS, vol. 27, No. 9, Aug. 1953.

[3] These materials correspond to the rubbers designated as R-1, R-2, and R-6, respectively, in the report cited in footnote 1.

one mixture. When these various factors are held constant in a given series of tests, however, differences in the load-deformation relation as between different mixtures may be used to compare the flow properties of the mixtures.

For example, the leading slope of the load-deformation curve shown in figure 1 is assumed to reflect the flow characteristics of the compacted specimen of bituminous mixture when tested in compression without lateral restraint. The curve shown in this figure is a typical one for mixtures tested at 77° F., and the basic elements as they were used in analyzing the curves for this report are as indicated in the figure. Some mixtures, however, are so plastic that no straight-line segment is obtainable. This latter type of curve is very often obtained in testing mixtures at temperatures of 120° or 140° F. In any case, a load component equal to one-third the maximum load is laid off on the middle section of the curve. The projection of this segment on the horizontal axis is carefully measured, and this quantity is taken as the deformation component. Unit vertical load (pounds per square inch of end-area of the test specimen) divided by the unit vertical deformation (inches per inch of specimen height) is termed the modulus of plastic deformation.

The computation, using the load-deformation curve in figure 1, obtained with a specimen 3 inches in diameter and 3 inches high, is as follows:

$$\text{Unit load} = \frac{\text{load (lb.)}}{\text{cross-sectional area of specimen (sq. in.)}} = \frac{577}{7.069} = 81.62 \text{ lb. per. sq. in.}$$

$$\text{Unit deformation} = \frac{\text{deformation (in.)}}{\text{height of specimen (in.)}} = \frac{0.026}{3} = 0.00867 \text{ in. per in.}$$

$$\text{Deformation modulus} = \frac{\text{unit load}}{\text{unit deformation}} = \frac{81.62}{0.00867} = 9{,}420 \text{ lb. per sq. in.}$$

A steep slope of the curve (high numerical deformation modulus value) indicates a mixture having stiff, hard, or even brittle properties; a shallow slope (low numerical modulus value) indicates a very plastic, easily deformed mixture. In the study of temperature susceptibility, temperatures of 77° and 140° F. were used in the unconfined compression tests.

In the laboratory study of blends of asphalts and rubbers [4] relative temperature susceptibility of the blends was based on a comparison of the slopes of lines connecting points plotted on semi-log graph paper to show penetration of the blends versus test temperatures. The numerical values for the slopes of such lines may be calculated from the following equation:

$$\text{Slope} = \frac{\log P_2 - \log P_1}{T_2 - T_1}$$

where P_1 and P_2 are the penetrations of the blends at the two temperatures T_1 and T_2.

It is generally agreed that the strength of a compacted bituminous mixture tested in unconfined compression at 77° F. is largely a function of cohesion. In turn, cohesion of the mixture tested at that temperature is largely a function of the viscous properties (penetration, melting point, ductility) of the bituminous cement. The compressive strength of a mixture containing a low-penetration asphalt is greater than a similar mixture containing a high-penetration asphalt. Increasing the temperature at which the unconfined compression test is made has the effect of reducing the viscosity of the asphalt with corresponding reduction in cohesive strength, and increase in plastic flow.

Therefore, since the temperature susceptibility of bituminous binders or blends of such binders and rubbers can be measured by the log penetration-temperature relation for such materials, it is logical to assume that temperature susceptibility of paving mixtures containing such binders, or blends of binders and rubbers, may be measured similarly by evaluating the rate of change in compressive strength and plastic flow resulting from temperature variation. Since both compressive strength and plastic flow are functions of the deformation modulus adopted for this study, a temperature-susceptibility index, calculated as follows, was used to compare susceptibility of the various mixtures to temperature change:

$$\text{Temperature-susceptibility index} = \frac{\log DM_1 - \log DM_2}{T_2 - T_1} \times 10^4$$

where DM_1 and DM_2 are deformation modulus values obtained in compression tests at the temperatures T_1 and T_2, respectively, expressed in degrees F. The 10^4 factor is used simply as a convenience.

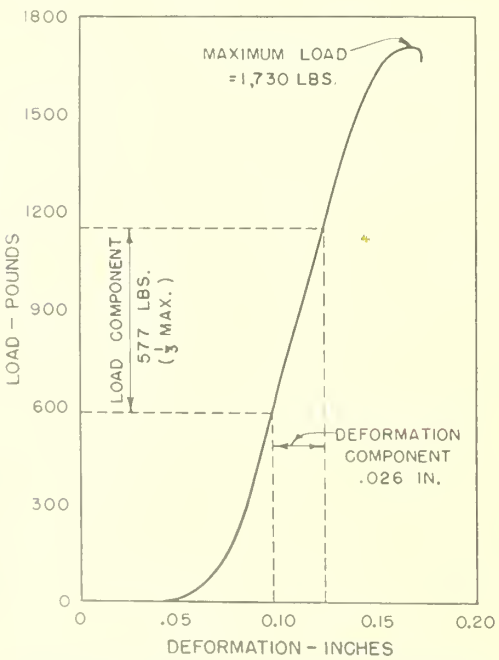

Figure 1.—Typical load-deformation curve and elements used in calculating deformation modulus.

In such comparisons, a relatively low index is construed as indicating a low temperature susceptibility—that is, a high resistance to temperature change. A high index is construed as indicating a high temperature susceptibility—that is, a low resistance to temperature change.

Differences in Density, Strength, and Temperature Susceptibility

Results of the unconfined compression tests made on specimens cured in an oven at 140° F. for 1 day, in conformity with standard compression-test procedure, are given in the left half of table 2. Each of the values shown is the average for three test specimens. In the first group of results shown in this table are a control mixture, and mixtures in the preparation of which reclaimed, synthetic, and natural rubber powders were added to the hot aggregate in the mixer before adding the asphalt. In molding test cylinders in this group of mixtures, the standard molding pressure of 3,000 pounds per square inch was used. This resulted in a wide range of densities for the several mixtures, with corresponding variations in values for compressive strength and plastic deformation. All the mixtures containing rubber powder were less compactible than the control mixture, and in general this observation based on tests of laboratory-prepared mixtures is consonant with the data available from tests previously made on a limited number of samples cut from newly laid experimental road installations. Table 3 shows data obtained on samples from three of these test roads. Of course, in studying these data, comparisons of bulk specific gravity are valid only as between mixtures of similar type.

In the first group of mixtures (the mixes with A in their designations) in table 2, in which powdered rubber was added to the aggregate and asphalt in the mixer, and in which the standard molding pressure was used, mix 1A (the control) was the most stable as judged by the compressive strength values at either test temperature, with the rubber mixes 3A (synthetic), 2A (reclaimed), and 4A (natural) following in descending order. The deformation modulus values at either test temperature showed the same rating for the mixtures in this group. Temperature-susceptibility ratings also had the mixtures in the same order, with the less stable mixtures being most susceptible.

These temperature-susceptibility evaluations of the mixtures were in disagreement with what might have been expected on the basis of results of tests made by a number of laboratories, including that of the Bureau of Public Roads, on blends of asphalt and rubber alone. Based on comparisons of the slopes of the log penetration-temperature curves mentioned in the previous discussion, it has been shown that the temperature susceptibility of blends of some rubbers and some asphalts is lower than that of the respective base asphalts.[5]

With a view to ascertaining to what extent this apparent discrepancy was attributable to

[4] See report cited in footnote 1, p. 92.

[5] See report cited in footnote 1.

Table 2.—Results of unconfined compression tests of bituminous concrete mixtures after exposure in 140° F. oven for 1 day and 21 days

Mix No.	Type of rubber	Form of rubber	Molding pressure	Tests after 1 day					Tests after 21 days					
				Density	Tests at 77° F.		Tests at 140° F.		Temperature susceptibility index	Density	Tests at 77° F.		Tests at 140° F.		Temperature susceptibility index
					Compressive strength	Deformation modulus	Compressive strength	Deformation modulus			Compressive strength	Deformation modulus	Compressive strength	Deformation modulus	
			Lb./sq.in.	*Gm./cc.*	*Lb./sq.in.*		*Lb./sq.in.*			*Gm./cc.*	*Lb./sq.in.*		*Lb./sq.in.*		
1A	None		3,000	2.240	245	8,830	69	4,190	51	2.240	315	10,150	87	4,650	54
2A	Reclaimed	Powder	3,000	2.120	149	3,500	21	655	115	2.121	178	3,830	37	995	93
3A	Synthetic	do	3,000	2.204	199	5,925	44	1,885	79	2.208	275	7,310	73	2,710	68
4A	Natural	do	3,000	2.058	134	2,500	14	345	136	2.057	164	2,620	23	485	116
1B	None		200	2.053	80	4,725	6	380	174	2.059	131	7,590	14	890	148
2B	Reclaimed	Powder	1,000	2.061	129	4,260	12	560	140	2.064	156	4,420	24	815	116
3B	Synthetic	do	400	2.061	126	7,010	13	690	160	2.061	173	8,770	29	1,320	130
2C	Reclaimed	Preblend	3,000	2.104	175	4,900	23	825	123	2.106	229	5,250	50	1,600	82
3C	Synthetic	do	3,000	2.217	261	7,590	61	3,820	47	2.217	317	10,130	90	4,900	50
4C	Natural	do	3,000	2.228	257	13,640	102	7,165	44	2.232	368	17,810	124	6,540	69
5C	Plasticized	do	3,000	2.237	320	11,450	92	6,270	42	2.237	400	17,520	125	8,220	52

the fact that the compacted mixtures varied widely with respect to density, tests were made on a second group of specimens (group B) in the preparation of which the molding pressure was varied so that the resulting densities would be approximately equal. Since mix 4A (natural rubber) in the first test group had the lowest density, molding pressures of 200, 1,000, and 400 pounds per square inch were used in preparing mixes 1B, 2B, and 3B, respectively, to bring the densities in these mixtures to approximately the same density as that of mix 4A.

With the density of the compacted mixtures removed as a variable, the compressive strength of mix 4A at either test temperature was highest. At 77° F. the compressive strengths for mixes 2B, 3B, and 1B in the equal-density group followed 4A in descending order. At 140° F. the compressive-strength rating position of mixes 3B and 2B were reversed, but mix 1B (control) again had the lowest rating.

Although the results of tests at 77° F. showed that mix 4A had slightly higher compressive strength than mix 1B, 2B, or 3B, they also showed that this mixture was the most plastic at that temperature, based on comparison of deformation modulus values. At this test temperature mix 3B was the least plastic, although it did not have the highest compressive strength. At 140° F., although the compressive strengths of mixes 4A and 3B were essentially the same, the deformation modulus value of mix 3B indicates that this mixture was twice as stiff (or half as plastic) as mix 4A. In this test group, all of the mixtures containing rubber powders were somewhat more resistant to temperature change than the control mixture for the group (1B). In comparisons limited to this group, it is thus possible to discern a degree of relation between the effect of adding rubber to asphalt as measured by laboratory tests of the blends of asphalt and rubber, and the effect as measured by laboratory tests of corresponding mixtures consisting of aggregate and the same asphalt and rubber when the rubber is added in the form of powder. The higher compressive strengths and lower temperature-susceptibility index values obtained for mixtures 2B (reclaimed), 3B (synthetic), and 4A (natural), as compared to the control mix (1B), reflect to a perceptible degree the reduction in penetration, increase in softening point, and reduction in temperature susceptibility noted in the results of tests made on the same asphalt and the same rubbers in the form of a blend.

Table 4 is arranged to facilitate the foregoing comparisons.

In studying the results of tests on the equal-density mixtures, it should be borne in mind that their importance is perhaps only academic, and lies chiefly in emphasizing the difference in effectiveness between adding the rubber to the mixture in the form of powder and adding it as part of the binder. In order to obtain the effects noted it was necessary, in molding the specimens of mixtures in which the rubber had been added in powder form, to compensate for the variations in compactibility characterizing the several mixtures by arbitrarily using molding pressures which, in two cases at least, would be considered wholly unrealistic.

From the foregoing discussion it is apparent that, under conditions of standard laboratory compaction, mixtures containing rubber that has been added to the aggregate in the form of powder are less compactible, are more plastic and less stable at both 77° and 140° F., and, within the temperature range 77°–140° F., are more susceptible to temperature changes than comparable control mixtures containing no rubber. Furthermore, in order to develop favorable comparisons as between the mixtures containing such rubber powders and the control mix, it was necessary to eliminate the pronounced compactibility differences by using unequal and unrealistic compactive pressures in molding the specimens for test.

Table 3.—Density of samples cut from roadway test sections

Type of pavement and rubber	Form of rubber	Density at time of construction		Density at subsequent testing	
		Year constructed	Bulk specific gravity	Year tested	Bulk specific gravity
Bituminous concrete (Tex.):					
None		1949	2.19	------	------
Natural rubber	Powder	1949	2.04	------	------
Reclaimed rubber	do	1949	2.15	------	------
Sand asphalt (Va.):					
None		May 1949	1.81	1951	2.17
Natural rubber	Powder	May 1949	1.71	1951	2.12
None		Sept. 1949	(1)	1951	1.99
Reclaimed rubber	Powder	Sept. 1949	(1)	1951	1.99
Sand asphalt (D. C.):					
None		1951	2.20	1952	2.23
Natural rubber	Powder	1951	2.00	1952	2.19
Synthetic rubber	do	1951	2.12	1952	2.20
Reclaimed rubber	do	1951	2.04	1952	2.20
Plasticized rubber	Preblend	1951	2.10	1952	2.24
Sheet asphalt (D. C.):					
None		1951	2.11	1952	2.15
Natural rubber	Powder	1951	2.06	1952	2.13
Synthetic rubber	do	1951	2.02	1952	2.06
Reclaimed rubber	do	1951	2.03	1952	2.10
Plasticized rubber	Preblend	1951	2.07	1953	2.17

[1] No data available regarding density as constructed.

Rubber More Effective When Preblended

The binders used in mixes 2C, 3C, and 4C consisted of blends of asphalt and reclaimed, synthetic, and natural rubber, respectively, prepared as has already been described. The binder in mixture 5C consisted of a blend of plasticized rubber and asphalt. Since the test specimens for the C group of mixtures were formed using 3,000 pounds per square inch molding pressure, test results for this group are directly comparable with those for the A group of mixtures.

The effect of the method of adding the rubber on the test results of the mixtures is

clearly apparent in the left half of table 2, in the higher densities obtained for the mixes containing preblended synthetic and natural rubber (3C and 4C) than were obtained for the corresponding mixes (3A and 4A) in which the rubbers were introduced in the form of powder. In the case of reclaimed rubber, this improvement in compactibility was not observed.

Similarly, the compressive strengths and the temperature-susceptibility index values show the relative effect of the two different methods of preparation, with higher stability and higher resistance to temperature change being obtained for preblended synthetic and natural rubber mixtures as compared with their rubber-powder counterparts. Although the stability of the preblended reclaimed rubber mix (2C) at both test temperatures was higher than that obtained with the same rubber in powder form (2A), the temperature susceptibility was about the same for this rubber regardless of the method of preparation.

Comparisons confined to the mixes comprising the C group and their control mix (1A) serve to indicate the relative value of the four different types of rubber with regard to their effect on the test properties of the mix after 1 day of oven curing. In the tests at 77° F. in the C group of mixtures, the compressive strength of mix 5C (plasticized) was highest, with the values for mixes 3C (synthetic), 4C (natural), 1A (control), and 2C (reclaimed) following in descending order. Comparative values for deformation modulus indicate that at this temperature mix 2C was the most plastic and 4C the least plastic. At 140° F., the compressive strength of mix 4C was highest with mixes 5C, 1A, 3C, and 2C following in descending order. At this temperature, as at 77° F., mix 4C was the least plastic and 2C the most plastic, based on deformation modulus values. Judged by the temperature-susceptibility index values, mix 2C was most susceptible to temperature change. Comparisons of index values indicate that mixes 3C, 4C, and 5C were slightly more resistant to the change in temperature than mix 1A.

As has been mentioned previously in discussing the results of compression tests of the equal-density (B) group of mixes, tests on blends of asphalt and rubber have indicated that in some cases the addition of the rubber resulted in lower penetration and reduced temperature susceptibility. In the comparison of mixes 2C, 3C, 4C, and 5C with the corresponding control mix 1A, it is possible to discern some relation between the changes observed in the tests of asphalt and rubber blends, and the slight changes observed in tests of the mixtures of aggregates and corresponding blends. That is, the higher compressive strengths of mixes 3C (synthetic), 4C (natural), and 5C (plasticized), as compared to the control mix 1A, are consistent with the higher viscosity resulting from blending corresponding rubbers with asphalts. Likewise, the reduction in temperature susceptibility resulting from blending rubbers and asphalt is reflected in the lower temperature-susceptibility index values obtained for mixes 3C, 4C, and 5C as compared to that of the control mix 1A. Reference is again made to table 4, in which related materials are compared directly.

The data in the left half of table 2 indicate, nevertheless, that although the rate of change in plastic properties as measured by temperature-susceptibility index values is slightly lower for the mixtures containing preblended rubber and asphalt than for the control mix without rubber, the mixes containing the rubber preblends are not substantially more plastic at lower temperatures and at the same time substantially less plastic at higher temperatures than mixes without rubber, at least in the 77°–140° F. temperature range. Deformation modulus values indicate that mixes 2C (reclaimed) and 3C (synthetic) were more plastic at 77° F. than mix 1A, but at the same time they were more plastic at 140° F. On the other hand, while mixes 4C (natural) and 5C (plasticized) were less plastic at 140° F. than the control mix, they were less plastic also at 77° F. The indications implicit in these test results are not in agreement with claims that paving mixtures containing rubber are more plastic at low temperatures and less plastic at high temperatures than similar mixtures without rubber.

Table 5.—Changes in compressive strength, deformation modulus, and temperatu susceptibility resulting from 20 additional days exposure to air in oven at 140° F.

Mix No.	Type of rubber	Form of rubber	Tests at 77° F.		Tests at 140° F.		Temperature susceptibility index [1]
			Compressive strength	Deformation modulus	Compressive strength	Deformation modulus	
			Percent	*Percent*	*Percent*	*Percent*	*Percent*
1A	None		+29	+15	+26	+11	+6
2A	Reclaimed	Powder	+19	+9	+76	+52	−19
3A	Synthetic	do	+38	+23	+66	+44	−14
4A	Natural	do	+22	+5	+64	+41	−15
1B	None		+64	+61	+134	+134	−15
2B	Reclaimed	Powder	+21	+4	+100	+45	−17
3B	Synthetic	do	+37	+25	+123	+91	−19
2C	Reclaimed	Preblend	+31	+7	+117	+93	−33
3C	Synthetic	do	+21	+33	+48	+28	+6
4C	Natural	do	+43	+31	+22	−8	+57
5C	Plasticized	do	+25	+53	+36	+31	+24

[1] Negative values indicate a gain in resistance to temperature change.

Effect of Prolonged Oven Exposure

The right half of table 2 gives the results of tests made on molded specimens that were held in a 140° F. oven for 21 days. These specimens were molded and placed in the oven at the same time as those represented in the left half of the table, the oven-exposure period for which had been 1 day in conformity with the usual procedure for the unconfined compression test. Since the densities of corresponding sets of specimens were in close agreement, differences in other physical properties as between 1-day and 21-day oven exposures may be fairly attributed to the effect of the extended heat treatment. Table 5 was prepared to show, in percentage form, the changes in numerical values for the several properties evaluated.

In the control mixture specimens molded at 3,000 pounds per square inch (1A), extended exposure to warm air resulted in higher compressive strengths and modulus values at both test temperatures. However, it is interesting to note that the temperature susceptibility of this mixture was little changed by the extended heating. The specimens containing this same mixture, molded at 200 pounds per square inch (1B) showed pronounced percentage increases in compressive strengths and modulus values. As shown in table 5, changes occurred in all of the mixtures containing rubber powders regardless of the pressure used in the molding operation. Changes in compressive strength and modulus values appear to be more marked in the results of tests made at 140° F. In studying the values given in table 5, it should be realized

Table 4.—Characteristics of asphalt AC–2 and blends of 5 percent rubber powders with this asphalt compared to characteristics of compacted mixtures of aggregate and these same materials

Type of rubber	Characteristics of rubber-asphalt blends [1]				Equal-density mixtures, with powdered rubber [2]		Equal-molding-pressure mixtures, with preblended rubber [2]	
	Rubber identification	Penetration at 77° F.	Softening point	Temperature susceptibility [3]	Mix. No.	Temperature susceptibility index	Mix. No.	Temperature susceptibility index
			° F.					
None		87	122	226	1B	174	1A	51
Reclaimed	R–2	80	126	207	2B	140	2C	123
Synthetic	R–6	74	127	202	3B	160	3C	47
Natural	R–1	67	147	165	4A	136	4C	44

[1] From *The effect of various rubbers on the properties of petroleum asphalts*, by R. H. Lewis and J. Y. Welborn. Tables 7 and 8 (see pp. 69 and 70).
[2] Mixtures 1B, 2B, and 3B were molded under variable pressures to the same density as was obtained for mix 4A molded under 3,000 lb. per sq. in. pressure. Mixtures 1A, 2C, 3C, and 4C were molded under 3,000 lb. per sq. in. pressure.
[3] Slope of log penetration-temperature curves × 10^4.

Table 6.—Results of immersion-compression tests of bituminous concrete mixtures after exposure in 140° F. oven for 1 day and 21 days

Mix No.	Type of rubber	Form of rubber	Molding pressure	Tests after 1 day							Tests after 21 days						
				Characteristics of specimens as molded			After immersion at 120° F.				Characteristics of specimens as molded			After immersion at 120° F.			
				Density	Air voids	Compressive strength at 77° F.	Absorption	Swell	Compressive strength at 77° F.	Retained strength	Density	Air voids	Compressive strength at 77° F.	Absorption	Swell	Compressive strength at 77° F.	Retained strength
			Lb./sq. in.	*Gm./cc.*	*Percent*	*Lb./sq. in.*	*Percent*	*Percent*	*Lb./sq. in.*	*Percent*	*Gm./cc.*	*Percent*	*Lb./sq. in.*	*Percent*	*Percent*	*Lb./sq. in.*	*Percent*
1A	None		3,000	2.249	10.6	238	2.4	0.8	177	74	2.248	10.7	309	2.0	0.3	269	87
2A	Reclaimed	Powder	3,000	2.125	15.3	162	4.4	.7	107	66	2.121	15.4	210	3.5	(1)	179	85
3A	Synthetic	do	3,000	2.206	12.0	206	2.9	.8	162	79	2.207	11.7	289	2.8	.6	233	81
4A	Natural	do	3,000	2.061	17.8	133	4.9	0	101	76	2.055	18.0	173	4.0	(1)	134	77
1B	None		200	2.043	18.8	72	6.4	(1)	69	96	2.051	18.5	140	5.9	1.4	108	77
2B	Reclaimed	Powder	1,000	2.048	18.4	115	5.2	(1)	94	82	2.060	17.9	166	5.4	(1)	121	73
3B	Synthetic	do	400	2.058	17.9	127	5.3	(1)	96	76	2.064	17.6	157	5.4	(1)	130	83
2C	Reclaimed	Preblend	3,000	2.122	15.4	194	4.3	1.0	128	66	2.114	15.7	215	4.4	.4	187	87
3C	Synthetic	do	3,000	2.219	11.5	258	2.7	1.3	177	69	2.220	11.4	324	2.6	.9	251	77
4C	Natural	do	3,000	2.236	10.7	254	2.5	1.0	184	72	2.237	10.7	330	2.8	.9	266	81
5C	Plasticized	do	3,000	2.235	10.9	303	2.7	1.1	207	68	2.238	10.8	391	2.4	1.0	280	72

[1] No swell.

Table 7.—Results of static immersion stripping tests

Mix No.	Type of rubber	Form of rubber	Estimated area remaining coated after 24-hour immersion at temperature indicated		
			100° F.	120° F.	140° F.
			Percent	*Percent*	*Percent*
1	None		90	85	75
2	Reclaimed	Powder	90	80	70
3	Synthetic	do	90	75	50
4	Natural	do	80	75	60
5	Reclaimed	Preblend	95	65	40
6	Synthetic	do	85	70	30
7	Natural	do	100	85	80
8	Plasticized	do	95	60	45

that increases in resistance to temperature change would be indicated by negative percentage values in the temperature-susceptibility index column, and lowered resistance to temperature change by positive values, since in table 2 high index values indicate high susceptibility to temperature change and low index values indicate low susceptibility.

All of the mixtures containing rubber added in powder form gained in resistance to temperature change during the 21-day exposure period. This could be construed as indicating that the full effect of rubber added in this manner may not be obtained immediately after construction of the pavement, and that some improvement in resistance to temperature change could be expected with the passage of time. Nevertheless, it should be noted that, according to the test results in table 2, the control mixture molded at 3,000 pounds per square inch was superior in resistance to temperature change to any of the mixtures containing rubber powder even at the end of the exposure period.

The effect of extended heating on the group of mixtures containing preblended rubber was quite variable. Marked increase in compressive strength at both test temperatures was noted for all four preblended mixes, but at 140° F. the change in values of the deformation modulus ranged from a 93-percent increase for the reclaimed rubber (2C) to an 8-percent reduction for the natural rubber mix (4C). The changes in temperature susceptibility varied from marked improvement in resistance to temperature change for the mixture containing reclaimed rubber (2C) to regression for the mixtures containing preblended natural, synthetic, and plasticized rubber.

Resistance of the Mixtures to Water Action

The results of immersion-compression tests made on the mixtures after 1-day exposure in the 140° F. oven are given in the left half of table 6. The retained strength values for the mixes containing synthetic and natural rubber in powder form (3A and 4A) were slightly higher than those for the corresponding control mix (1A). Retained strength values for all of the preblended rubber mixes (2C, 3C, 4C, and 5C) were lower than for the control mix. In the equal-density group, all of the mixes containing rubber (2B, 3B, and 4A) showed consistently lower retained strength values than the control mix (1B).

Results of immersion-compression tests made on companion specimens kept in an oven at 140° F. for 21 days after molding are given in the right half of table 6. Judging from the retained strength values, extended oven exposure increased the resistance to water action of all the mixes except the control mix molded at 200 pounds per square inch (1B) and the reclaimed rubber mix molded to equal density (2B). The mix containing synthetic rubber powder in the equal density group (3B) showed a higher retained strength value than the corresponding control mix (1B). All the other mixes containing rubber showed retained strength values equal to or lower than the corresponding control mixes.

Static Immersion Stripping Tests

In table 7 are given the results of the static immersion stripping tests on mixtures of quartzite and asphalt, both with and without rubber. These results can be compared, qualitatively at least, with those for the immersion-compression tests on mixtures that had been kept in the 140° F. oven for 1 day, shown in table 6. No stripping tests were made on mixtures held in the oven for 21 days.

The results of the stripping tests at the 100° F. immersion temperature showed marked improvement over the control mix (No. 1) for the preblended natural rubber (No. 7) only. When added as a powder, the results at 100° F. indicate that natural rubber was the least effective. In the tests at 120° F., only the preblended natural rubber (No. 7) appeared to be the equal of the control mix (No. 1). At that temperature the plasticized rubber preblend (No. 8) appeared to be least resistant to stripping. In the tests at 140° F., the natural rubber preblend (No. 7) was the most resistant to stripping and the synthetic rubber preblend (No. 6) the least resistant. At that temperature the natural rubber preblend mix (No.7) was the only other mixture as resistant to water action as the control mix (No. 1).

In general, the results of the static immersion stripping tests confirm the indications based on the results of the immersion-compression test: namely, that the addition of rubber to bituminous mixtures containing hydrophilic

Table 8.—Results of modified California abrasion test

Mix No.	Type of rubber	Form of rubber	Average loss [1] after exposure at 325° F. for—					
			0	¼ hr.	1 hr.	2 hrs.	4 hrs.	8 hrs.
1	None		0.23	0.03	0.02	0.03	0.16	0.57
2	Natural	Powder	.64	.40	.11	.19	.25	.44
3	do	Blended	0	0	0	.01	.09	.30
4	Synthetic	do	.81	.14	.20	.24	.29	.53
5	Reclaimed	do	.93	.30	.23	.35	.49	.83
6	Plasticized	do	.21	.09	.22	.29	.64	1.42

[1] In grams per 1,000 grams of shot.

aggregate was ineffectual in improving the resistance of such mixtures to the action of water.

Effectiveness of Rubber in Abrasion Test

In table 8 are given the results of the modified California abrasion tests on mixtures consisting of Ottawa sand and asphalt, with and without rubber. In the test made on mix 1, the loss occurring in the specimen that had not been exposed to the 325° F. temperature at all would be considered fairly high, although a certain amount of loss on similarly uncured or unexposed fresh mixtures is usual in this test.

A sharply reduced loss was noted for mix 1 in the specimen that had been exposed for ¼ hour at 325° F., and this improvement in resistance to abrasion conforms to the general pattern for similar mixtures containing asphalts having histories of good service behavior. This same pattern of relatively high loss for specimens consisting of fresh mixture and reduced loss after ¼-hour exposure was followed also by mixes 2, 4, 5, and 6, but the reductions were of different order. The deviation from this pattern shown by mix 3 was notable; there was no loss either in the fresh specimen or in the specimen exposed in the oven for ¼ hour. Indeed, the first measurable loss for this mixture was noted only after 2 hours at 325° F.

After 4 hours exposure, the abrasion loss on mix 1 increased sharply, and after 8 hours the loss amounted to 0.57 grams per 1,000 grams of shot. After 4 hours exposure mix 3 was most resistant to abrasion and mix 6 least resistant. After 8 hours exposure mix 3 was still the most resistant to abrasion, with mixes 2, 4, 5, and 6 following in descending order of resistance.

The results of this very limited number of tests indicate that the natural rubber, when added to the mixture as a blend of rubber and asphalt, increased the resistance of the mixture to abrasion. The marked difference in results between mix 2 and mix 3 offers additional evidence supporting the belief that to obtain maximum effectiveness from the rubber materials they should be preblended with the asphalt before mixing with the aggregate. The abrasion test results also indicate that the effectiveness of additive rubber in improving resistance of asphaltic mixtures to abrasion varies with the type of rubber.

U. S GOVERNMENT PRINTING OFFICE: 1954

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Express Highways in the United States: a Bibliography (1945).

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

FOR OFFICIAL USE

AS OF AUGUST 31, 1954

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$23,484	$7,695	$4,241	180.6	$8,693	$4,224	206.1	$44,509	$22,272	480.7	$60,897	$30,737	867.4
Arizona	10,647	5,096	3,332	94.4	3,592	2,557	79.6	5,207	3,728	94.3	13,895	9,617	268.3
Arkansas	15,750	9,548	5,575	401.6	4,359	2,189	139.2	17,144	8,479	415.4	31,051	16,243	956.2
California	37,104	19,275	10,426	127.4	19,733	10,316	51.0	106,721	50,780	327.9	145,729	71,522	506.3
Colorado	15,386	9,259	5,262	183.4	3,537	1,965	93.0	18,629	10,163	167.9	31,425	17,390	444.3
Connecticut	18,656	1,476	738	4.9	4,346	2,025	6.2	5,151	2,584	12.7	10,973	5,347	23.8
Delaware	4,704	1,140	745	8.3	4,686	2,145	3.3	7,241	3,948	42.2	13,367	6,838	53.8
Florida	18,339	18,525	9,392	282.2	6,541	3,112	101.9	19,387	10,188	303.4	44,453	22,992	687.5
Georgia	25,764	18,134	9,248	459.8	11,031	5,483	287.4	39,725	18,691	585.1	68,890	33,422	1,332.3
Idaho	8,931	6,529	4,176	112.4	3,235	2,000	53.8	15,734	9,804	271.3	25,498	15,980	437.5
Illinois	40,526	41,102	22,562	365.9	16,482	8,710	83.8	74,886	38,473	508.3	132,470	69,745	958.0
Indiana	27,113	33,325	16,649	141.6	24,635	13,726	83.1	26,543	13,598	162.9	84,503	43,973	387.6
Iowa	18,656	13,105	7,374	348.2	8,291	4,240	299.1	24,723	13,610	843.3	46,119	25,224	1,490.6
Kansas	19,916	12,139	6,152	792.7	7,196	3,592	352.6	15,659	7,968	750.0	34,994	17,712	1,895.3
Kentucky	17,198	11,418	6,156	69.5	7,504	3,818	102.4	25,728	13,180	240.4	44,650	23,154	412.3
Louisiana	18,080	15,002	7,189	114.1	8,374	3,385	12.2	28,786	13,985	144.6	52,162	24,859	270.9
Maine	6,970	9,544	5,081	58.0	2,182	1,092	28.0	13,165	6,535	99.4	24,891	12,715	185.4
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Nevada	14,762	2,643	2,264	83.2	--	--	--	7,577	6,273	204.1	10,220	8,537	287.3
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Oregon	15,107	2,383	1,409	12.5	2,023	1,196	40.0	14,746	8,989	206.2	19,152	11,594	258.7
Pennsylvania	46,809	18,163	9,082	9.2	25,112	12,693	38.7	99,936	48,759	205.1	143,511	70,534	253.0
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Utah	8,752	3,934	3,024	69.2	1,145	1,060	39.4	9,737	7,422	116.8	15,086	11,506	225.4
Vermont	5,332	2,616	1,315	29.7	246	123	3.0	9,506	4,787	72.1	12,368	6,225	104.8
Virginia	19,564	13,807	6,692	219.9	7,480	3,372	105.7	24,055	11,307	157.6	45,342	21,571	483.2
Washington	15,368	10,428	5,476	113.8	4,062	2,026	60.7	21,838	11,635	183.8	36,328	19,137	358.3
West Virginia	14,916	9,923	5,013	44.8	3,060	1,546	27.5	15,263	7,616	38.7	28,246	14,175	111.0
Wisconsin	23,978	12,318	6,746	77.2	5,172	2,582	104.9	28,301	14,052	378.4	45,821	23,380	560.5
Wyoming	6,768	3,970	2,594	69.3	695	450	10.5	7,774	4,874	195.1	12,439	7,918	274.9
Hawaii	5,141	1,774	870	5.4	1,335	665	6.9	9,484	4,436	9.8	12,593	5,971	22.1
District of Columbia	8,679	5,853	2,932	4.0	3,361	1,070	.5	10,068	4,830	2.8	19,282	8,832	7.3
Puerto Rico	10,593	5,114	2,681	35.0	4,697	1,821	6.9	15,718	7,448	46.7	25,531	11,750	88.6
TOTAL	1,027,284	679,325	340,751	10,443.1	364,334	183,003	5,570.8	1,547,647	777,093	15,664.2	2,551,306	1,300,847	31,678.1

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

Contents of this publication may be reprinted. Mention of source is requested.

Motor-Vehicle-Use Studies in Six States

BY THE DIVISION OF RESEARCH
BUREAU OF PUBLIC ROADS

Reported by
THURLEY A. BOSTICK, Transportation Economist,
ROY T. MESSER, Highway Transport Research Engineer, and
CLARENCE A. STEELE, Chief, Taxation and Economic Studies Section

Up-to-date information about the characteristics of motor-vehicle ownership and use supplies some of the needs of highway administrators in the broad planning of highway programs. This report analyzes many of the findings that have recently become available from the first of the motor-vehicle-use studies conducted by the State highway departments since the close of World War II.

The primary purpose of the report is to indicate to those who may have use for such information the types of data that are available from the motor-vehicle-use studies and some of the principal relations that these data may best be used to demonstrate—such information as the distribution of vehicle ownership geographically and by occupational groupings, the characteristics of vehicle use, including annual travel, road systems used, places of travel, trip length, and travel purpose, the age distribution of vehicles, the transportation media used by workers traveling to work, and the age and sex of motor-vehicle operators.

A secondary purpose of the report is to provide factual information based on the findings of the studies conducted in the six States covered, but it must be remembered that these States—located in the north central and south central parts of the country—are not sufficiently well distributed to provide even an approximation of Nationwide coverage.

THIS REPORT presents the results of motor-vehicle-use studies conducted during the summer of 1951 in six States—Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin. These studies, and others of a similar nature that have since been undertaken, provide information about the characteristics of motor-vehicle ownership and use that is not available from the ordinary types of traffic surveys. Similar data for a number of these characteristics were obtained in studies made by most of the State highway departments between 1930 and 1940, but it was generally believed that this information was no longer fully representative of existing conditions.

Since this report is based on data obtained during a single season in 1951 in only six States (and some of the information was not available for all six), the presentations made must be considered as being subject to modification as more information of wider coverage and more recent date is received. Furthermore, because of their location, it should not be assumed that the findings of studies made in these few States are representative of the Nationwide situation. However, it is believed that the findings reported herein, especially those presented for individual population or occupation groupings, can be considered as being generally indicative, within reasonable range, of conditions that now exist.

SUMMARY OF FINDINGS

Motor-vehicle distribution.—Among the characteristics of motor-vehicle distribution revealed by these studies were the following:

Motor vehicles were owned or operated by residents of nearly three-fourths of all occupied dwelling units; the range was from 80 percent in unincorporated areas to 57 percent in cities of over 100,000 inhabitants. Motor vehicles were owned by 84 percent of all farm families.

Two or more automobiles were owned in 1 of every 15 households. Trucks only were operated by residents of 1 of every 14 households.

The average age of all automobiles in use in 1951 was 7.1 years; in 1941 it was 5.5 years. In 1951, 4 out of 10 vehicles in use were prewar models (built in 1941 or earlier).

Motor-vehicle operators.—Analyses of information reported about motor-vehicle operators showed the following significant characteristics:

Fifty-three percent of all persons 14 years or older were licensed to drive.

Two-thirds of the licensed drivers were males.

About one-fourth of the licensed drivers of both sexes were found in the 30-39 age group, more than in any other group.

Travel to and from work.—These studies, which covered all modes of home-to-work transportation, revealed that the automobile was the principal means used by gainfully employed persons for such travel. The highlights of the findings with respect to home-to-work transportation are as follows:

Sixty-eight percent of all workers used transportation or walked between their homes and places of work; 32 percent worked at their place of residence (91 percent of the farmers and farm managers lived on the farms they operated).

Sixty-one percent of the workers who used transportation or walked to and from their jobs used automobiles for all or part of the trip.

Even in the group that lived less than 1 mile away, nearly one-half of the workers traveled to work by automobile. For those traveling greater distances, 67 to 85 percent traveled to work by car, depending on the distance traveled.

One out of six workers who used transportation or walked to work walked the entire distance; 85 percent of these lived closer than 1 mile. Beyond 1 mile from place of employment, walking ceased to have significant influence as a mode of travel to work.

Only in cities of over 100,000 inhabitants did more than one-third of all workers use public transit as a means of travel to work. Even in these large cities more workers traveled by auto than by public transportation.

Distribution of automobile travel.—These studies provided the distributions of automobile travel for residents of the State by place of travel, according to the place of residence and occupation of the principal operator.

Approximately one-half of the travel by both rural residents and residents of incorporated places was on the rural primary roads.

Travel on the local rural roads was mostly by the rural residents.

The percentage of travel on city streets by urban residents varied in direct relation to the size of the city of residence.

Trips confined entirely within one city or contiguous cities comprised one-half of the automobile travel within incorporated places by all residents of the State. In the large cities this proportion reached 75 percent.

Only 8 percent of all travel occurred outside the State of residence. The range by occupation was from under 3 percent for farmers to about 11 percent for professionals, proprietors, and clerks.

Farmers and farm managers used the local rural roads more and city streets less than any other occupational group.

There was relatively little difference in the total use of primary State highways by occupational groups. The range was from 50 percent for farmers to 61 percent for the professional group.

Automobile trip lengths.—The pattern of highway use by passenger cars was found to be composed predominantly of short-length trips.

Six of every ten one-way trips were under 5 miles in length. Four-fifths of the trips were less than 10 miles in length. The average one-way length of trip was 8.3 miles.

One-half of the total travel resulted from trips of under 30 miles one-way length.

Trip-length frequency distributions were similar for residents of each population group and for the several occupational groups. Trip-length frequency distributions were also generally similar for all purposes of travel except vacations and, to a lesser extent, medical and dental.

Shopping trips were the shortest and vacation trips the longest, averaging 3.7 and 250 miles, respectively.

Purpose of travel.—Automobile travel was analyzed according to purpose for which the trips were made. Some of the significant relations which were found in these analyses are outlined below.

Sixty percent of the travel and 73 percent of the trips by passenger cars were for the purpose of "necessity" travel (earning a living or family business). About three-fourths of this travel and two-thirds of the trips were made in connection with earning a living. One-half of the travel and two-thirds of the trips made in connection with earning a living resulted from travel to and from work.

Both rural and urban residents performed about 45 percent of their total automobile travel in connection with earning a living.

Family business and educational and related travel accounted for a greater proportion of the total travel by residents of unincorporated areas (25 percent) than it did for residents of incorporated places (14 percent). On the other hand, a higher proportion of travel for social and recreational purposes was found for the residents of incorporated places (43 percent) than of unincorporated areas (29 percent).

"Necessity" travel—for earning a living or family business—averaged between 55 and 65 percent for gainfully employed workers in most occupational groups.

Comparison with earlier studies.—It was possible to make certain comparisons of automobile travel between the motor-vehicle-use studies and the earlier road-use studies in four of the States. In spite of a major shift in the classification of rural mileage in Louisiana, the findings of the studies were remarkably consistent:

Although the total travel on each class of roads and streets was considerably greater in 1951 than in 1936, the proportion of travel on city streets to total travel decreased from 35 to 27 percent.

For residents of incorporated places, the travel distribution showed a decline in travel on city streets from 43 percent in 1936 to 36 percent in 1951. For residents of unincorporated areas, the travel distribution by place of travel in 1951 was almost identical to that of the earlier studies.

DESCRIPTION AND STATUS OF STUDIES

Statistical studies of motor vehicle use are not new. The present projects are but the latest in a series of similar investigations that have taken place over a period of more than 20 years. The studies have all had a common basic set of purposes: to learn more about the characteristics of motor-vehicle ownership and use than can be learned from the ordinary types of traffic surveys. These purposes may be stated in more detail as follows:

1. To learn how the ownership of automobiles and trucks is distributed geographically among rural residents and city dwellers, and among various occupational groupings.

2. To determine the characteristics of the use made of private and commercial motor vehicles by the various ownership groups mentioned above. These include estimates of annual travel of automobiles and trucks, road systems used and places of travel, and trip length and purpose of travel.

3. To determine the age distribution of the automobiles owned by various segments of the population, and the age and weight distribution of the commercial vehicles owned by the major groups, such as farmers and retail establishments.

4. To learn something about the rates of fuel consumption of motor vehicles, classified according to residence of owner, vehicle type, and age groups.

5. To obtain information about the transportation media used by gainfully-employed workers in getting to and from work.

Such information has many uses both to government agencies and to business and industry. Perhaps its most important uses are in connection with the development of long-range plans for the construction and reconstruction of the several systems of roads in a given State, especially the financial and economic aspects of such programs. Data of these types are especially valuable in determining equitable bases for the apportionment to highway purposes of motor-vehicle-user tax receipts. They have also been helpful in other State and Federal legislative programs.

In times of national emergency such data are essential in the determination of the allocations of critical materials and equipment for the construction and maintenance of highways and the allocation of critical materials, such as steel and rubber, to industry for the manufacture of parts, tires, and accessories to keep vitally needed vehicles in service. During World War II such information was also used in determining the bases upon which motor fuel and tires were rationed to the public.

Industry, especially in the automotive field, finds information obtained from such studies useful in planning and developing production and markets. From time to time various industry groups have made studies of the motor vehicle and its usage to meet particular needs. Since such studies have been limited in their coverage, these groups have been especially anxious to have the benefit of the more inclusive data developed from the motor-vehicle-use studies.

Description of Present Studies

The present motor-vehicle-use studies combine features from several types of earlier studies, notably the road-use and motor-vehicle-allocation studies made during the period from 1935 to 1940; the war-industry-transportation studies made during World War II; and the metropolitan area origin-and-destination studies that have been in progress since the end of the war. They are sampling studies in which the sample base is the number of dwelling-units, which is believed to be the best available base for such a study. It has the advantage of being a universal base for which reasonably current statistics are available for the Nation, the individual States, and various population groupings or individual places within the States. It also has the advantages of being more stable, less inflated, and more readily measurable than are motor-vehicle registrations, which were used as the sampling base for the road-use and motor-vehicle-allocation studies. (Motor-vehicle registrations cover the total registered in a whole year, rather than the number of vehicles in use at a particular time.)

The samples, which are scientifically drawn, are relatively small—often as small as one-half of 1 percent in some population groups—but large enough to provide reasonable stability in the values sought on a Statewide population-group basis. In some States, where supplementary data on trucks are needed, separate truck samples are drawn. Interviewers are sent to call upon the residents of each dwelling unit chosen in the sample. The respondents are asked to provide the following information:

1. Listing of residents of the dwelling unit, indicating sex, approximate age, and whether or not those of legal driving age possess vehicle operators' permits;

2. Occupational information about those residents who are gainfully employed, distance to place of work, and means of getting to work;

3. Inventory of automobiles and trucks regularly driven by residents of that dwelling unit;

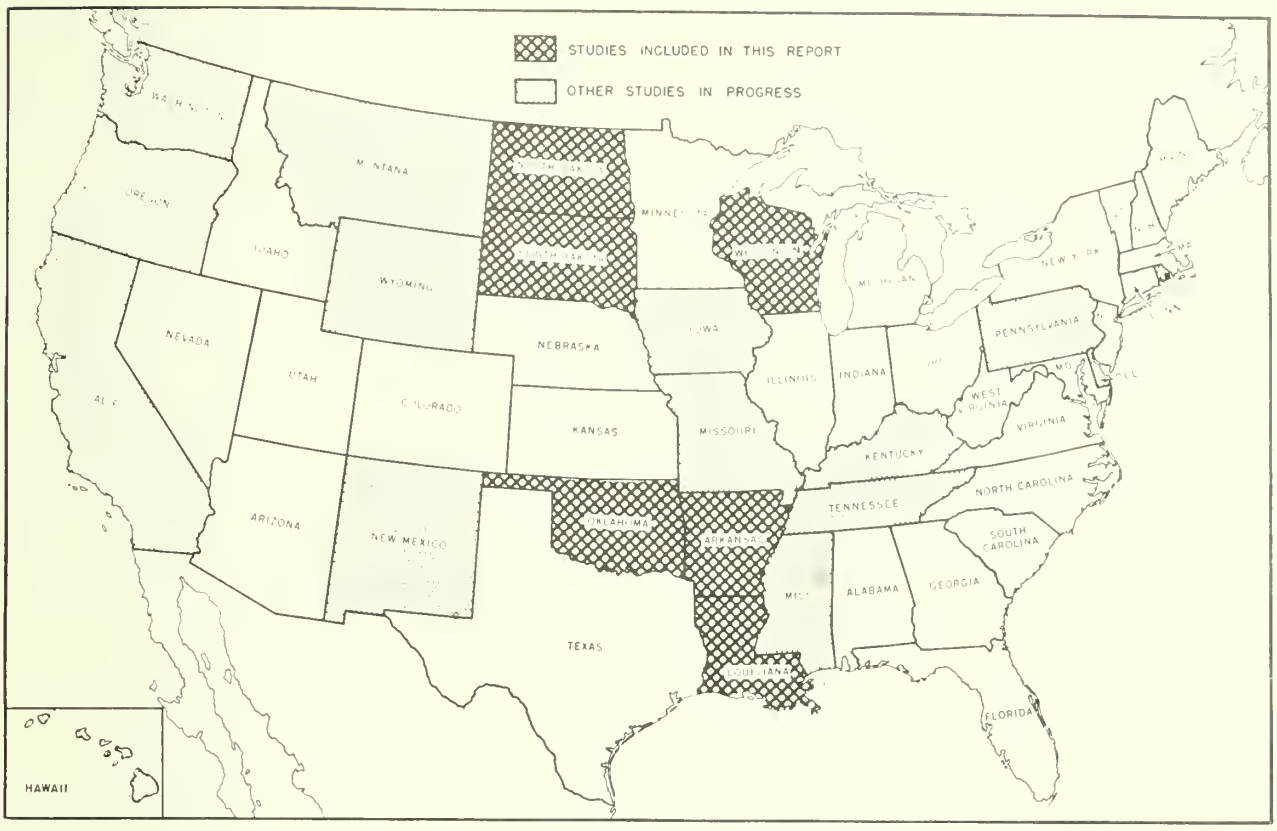

Figure 1.—States participating in the motor-vehicle-use studies; August 1954.

4. Estimated total annual travel of vehicles reported upon, and the fuel consumed by them;

5. Description of motor-vehicle trips made by residents of the dwelling unit, ending on certain days, on which a resident was the driver of the vehicle. Information sought includes the origins and destinations of such trips, purposes, routes used, trip length, and number of persons carried in the vehicle. In the case of truck trips, the general description and weight of commodities hauled is also sought.

The procedure recommended for the motor-vehicle-use studies provides for full-year coverage of ownership and travel data by dividing the sample to be obtained in each population group into four equal segments, one to be taken each season. By this means the effects of the seasons upon characteristics of travel will be properly reflected for each population group. A further indication of the annual character of long-distance travel is obtained by asking the respondents to provide information about all trips of more than 7 days duration made during the last 12 months prior to the time of interview.

However, the complete 1-year study described above has not been made in all States where the motor-vehicle-use study has been undertaken. Where the sampling covered only one or two seasons, special analysis procedures had to be designed to expand the data obtained to an annual basis. Where all four seasons were covered, the analysis of the travel and related data obtained involved a straightforward expansion by which the level of each seasonal segment of the sample was raised to the level of the universe for that season by the application of expansion factors reflecting the relation of the sample to the universe. The combination of the seasonal data to obtain figures for a year is a relatively simple process.

Unlike the ordinary traffic survey, the motor-vehicle-use study is not intended to obtain any information about the use of the highways of the State being studied by vehicles not registered or kept there. It does, however, provide information about total vehicle use—in and outside of State of residence—of private and commercial vehicles that is not obtained in the ordinary traffic survey. Also, the motor-vehicle-use study provides information about the rural-urban breakdown of trips that is not obtained in the traffic survey.

The studies were conducted by the State highway departments as highway planning projects, with the technical cooperation of the Bureau of Public Roads. The highway departments have used great care in selecting the enumerators who contacted the respondents, and public reaction to the study has been very good. In two instances, the highway departments have enlisted the services of the United States Bureau of the Census in organizing and conducting the field work; in all other instances the enumerators have been employees of the highway departments.

Status of Motor-Vehicle-Use Studies

In August 1954, motor-vehicle-use studies had been completed or were then in progress in 20 States and in the Territory of Hawaii. Their location and status are shown graphically in figure 1. Thirteen of these studies were of the full, extended type; i. e., they cover the four seasons of an entire year. Three were of the modified-extended type where the size of the sample taken was somewhat less than would be taken in a complete full-year study, and the sampling was confined to one or two seasons. The remaining five studies were of the limited type in which the size of the sample taken was approximately equal to that which would normally be taken in a single season, and the interviewing was completed within one season. The limited study is intended only to be used in connection with data obtained in other States in determining regional characteristics of vehicle ownership and travel; the samples taken are usually not large enough to provide stable statistics for the one State only.

In one State, South Dakota, the officials were so well pleased with the results of a limited study that they have since made a complete extended study.

Coverage of This Report

Although interviewing is complete and analysis complete or far along in a number of States, data for only a few have so far been furnished to the Bureau of Public Roads, and some of those reports are incomplete. Accordingly, it has been necessary to base this report upon the results obtained in only a few States, and the presentations made must be considered as being only preliminary and subject to modification as more information from other States is received. The data shown

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Table 1.—Dwelling-unit bases and motor-vehicle-use study samples in six States

State and type of study	Total dwelling units	Original estimate of dwelling units in sample	Effective sample			Incomplete contacts		Total contacts made
			Acceptable interviews	Vacant dwelling units	Total	Nonusable interviews	Refusals	
Arkansas (limited)	576,054	3,468	3,327	290	3,617	31	10	3,658
Louisiana (modified extended)	731,816	5,063	4,783	295	5,078	560	(1)	5,638
North Dakota (limited) [2]	177,980	441	326	19	345	1	----	346
Oklahoma (limited)	724,790	1,502	1,363	115	1,478	208	20	1,706
South Dakota (limited) [3]	195,079	523	504	9	513	9	1	523
Wisconsin (limited)	1,082,216	2,599	2,091	60	2,151	58	12	2,221
Total	3,487,935	13,596	12,394	788	13,182	867	43	14,092

[1] Included with nonusable interviews.
[2] North Dakota also conducted a full-scale road-use study.
[3] South Dakota has since made an extended motor-vehicle-use study, the results of which were not available at the time of this report.

Table 2.—Comparisons of total populations from Census estimates with estimates obtained from motor-vehicle-use study data in six States

State	Total population, 1950 Census	July 1, 1951, total population (Census estimate)	Estimated population, motor-vehicle-use study	Difference, motor-vehicle-use estimate from Census estimate
				Percent
Arkansas [1]	1,335,053	1,325,000	1,234,895	−6.8
Louisiana	2,683,516	2,747,000	2,641,265	−3.8
North Dakota	619,636	602,000	625,724	+3.9
Oklahoma	2,233,351	2,262,000	2,087,942	−7.7
South Dakota	652,740	647,000	633,759	−2.0
Wisconsin	3,434,575	3,481,000	3,680,043	+5.7
Total	10,958,871	11,064,000	10,903,628	−1.4

[1] Persons under 14 years of age excluded.

herein include the modes of travel used in going to and from work, the distribution of licensed motor-vehicle operators, the characteristics of motor-vehicle distribution, and the characteristics of travel, such as distribution and purposes of travel by systems and classes of places and length of trips. The data were developed from reports received from six States: Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin. As will be indicated in the text, some types of information were available for all six States; some for only four or five of the six.

Information relating to total annual travel and average annual mileage per vehicle is omitted from this report since the studies made by the six States were conducted in the summer season only.

Table 1 indicates the coverage of these studies on a Statewide basis, and shows the dwelling-unit universe, the original estimates of the samples to be obtained, the effective samples obtained, and the incomplete interviews reported. Because of the differing sample rates employed in the various population groups, it is not possible to calculate the effective Statewide sampling rate from these data.

The procedures recommended for the extended studies are designed to provide general coverage of the "universe" of private and commercial automobiles, trucks, and tractor-trucks domiciled in any State studied. The hypothesis upon which the sampling methods is based considers that through the dwelling-unit contacts a representative cross-section of operators of all classes of these vehicles, including drivers of fleet-operated vehicles, such as company-owned automobiles assigned to salesmen, and of over-the-road transport trucks, would be obtained. However, the total number of some of these classes of vehicles in operation in most States is relatively small, and the sampling rates ordinarily employed even in the extended studies are so low that it is not to be expected that the samples obtained will be large and stable enough to permit separate analyses to be made for such groups. It is for that reason that the supplementary truck studies previously mentioned are made.

In the case of limited studies, such as those reported upon here, the total truck samples obtained in some population groups were extremely small and their stability is open to question. Consequently, presentations of travel and trip-length information for trucks have been omitted from this report.

Reliability of Data

Although certain tests of the reliability of the information obtained in the motor-vehicle-use studies can be made by statistical estimates, perhaps the most reliable indications are found when values computed from the samples are compared with totals for those items that are known or can be readily approximated from other sources. For example, table 2 shows comparisons of known and estimated population totals for the six States for which data are presented in this report. It can be seen that the population estimates obtained by expanding the samples very closely approximated the estimates made by the Bureau of the Census for 1951. Other comparisons which have indicated a satisfactory level of reliability in the motor-vehicle-use study data include comparisons of total travel on rural highways with figures obtained from the regular traffic surveys, and comparisons of estimated vehicles in service with numbers of vehicles registered.

This report also contains comparisons of data obtained in these studies with like information derived from the prewar road-use studies, the reliability of which has been reasonably substantiated. The findings of these studies display marked similarity to those of the earlier studies.

Since all but one of six motor-vehicle-use studies reported on herein were of the limited type, and since the six States are not so located geographically as to provide even an approximation of Nationwide coverage, it was believed that it would be undesirable to place too much emphasis in this report upon the specific values found in these particular studies. Such data might be taken to have Nationwide implications which may not be substantiated, to the same degree at least, as further data become available. For that reason the presentations that follow are limited largely to percentage distributions. These yield a clear indication of the relations involved, which probably will not be greatly modified by the findings of studies completed subsequently, although even this is conjectural.

MOTOR-VEHICLE OWNERSHIP

One of the major purposes in making the motor-vehicle-use studies is to learn how the ownership or situs of automobiles and trucks is distributed geographically and among various economic groupings within the population. This report presents preliminary information on these subjects that have been obtained in the studies made in the six States previously mentioned.

In these studies, the place of ownership is considered to be the place of residence of the regular driver of the vehicle. By far the greater number of vehicles reported upon were owned and operated by the respondents from their places of residence.

Distribution of Motor Vehicles

Figure 2 and table 3 indicate for the six States combined the percentage classification of occupied dwelling units, in total and by population groups, according to nature of motor-vehicle distribution. This same information for individual States with population groups combined is shown in figure 3.

It will be noted, in figure 2 and in many of the following figures, that data are represented for all places combined, then separately for unincorporated and incorporated places, and finally for each population group of the incorporated places.

From table 3, it may be observed that 27 percent of all dwelling units (households) for all population groups combined reported that neither cars nor trucks were owned. In the unincorporated areas only 20 percent of the dwelling units were without either automobiles

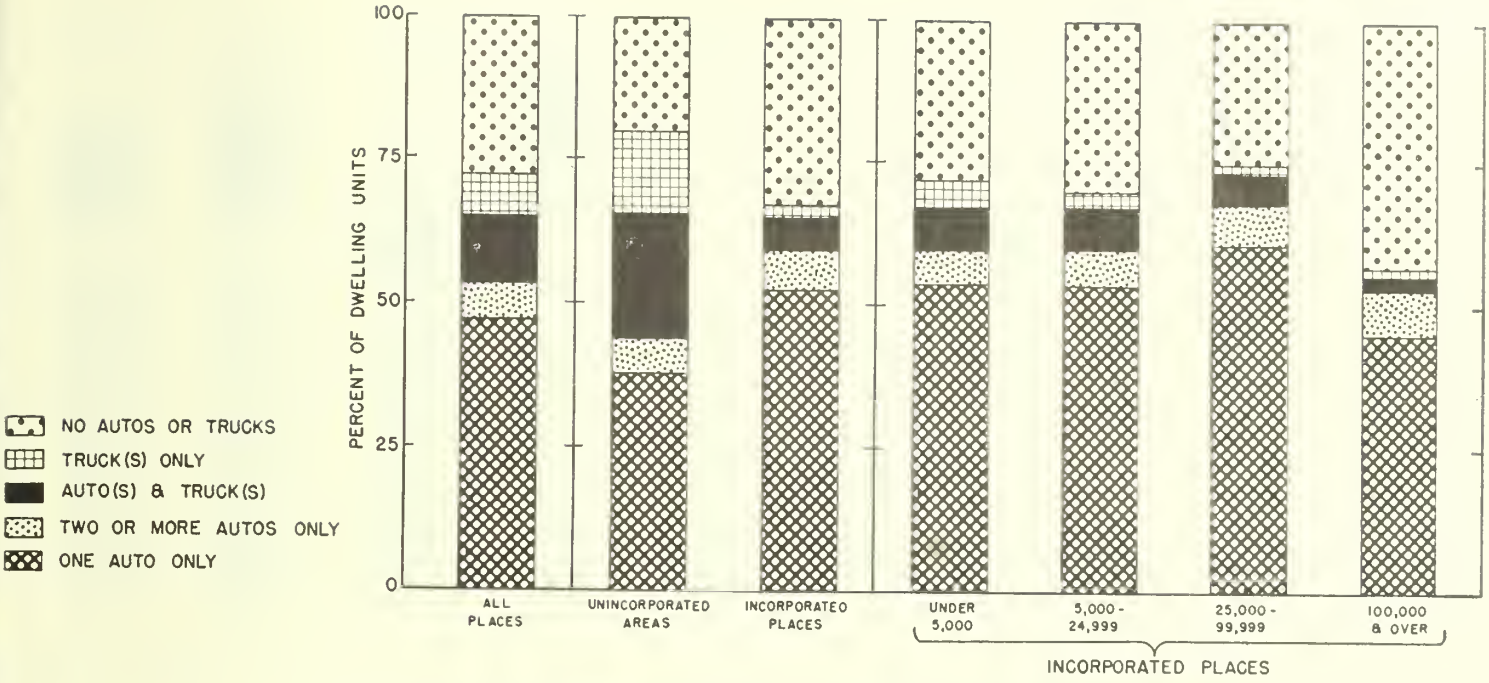

Figure 2.—Occupied dwelling units in each population group classified by number of vehicles operated by residents; summer of 1951, six States.

or trucks, while the cities of 100,000 inhabitants or more reported 43 percent. This points out the obvious—the essentiality of motor vehicles to those living in rural areas and the effects of adequacy of public transportation and the expense of motor-vehicle ownership upon residents of the largest cities.

Among the individual States, the range in dwelling units with no motor vehicles, as shown in figure 3, was from 4 percent in North Dakota to 45 percent in Louisiana.

Households reporting one automobile only and no other vehicles comprised 47 percent of the total for the six States, with a range of from 36 percent in Arkansas to 57 percent in South Dakota. The ownership of one automobile, and no other type of motor vehicle, was lowest in the unincorporated areas and highest in the medium-sized cities of 25,000–99,999 population.

Multiple-car ownership (without the accompanying ownership of trucks) did not bulk large in any population group, all appearing to cluster rather closely around the average for all groups of 7 percent of the total of all dwelling units. Here the range was from 4 percent in Arkansas to 8 percent in Wisconsin.

Cases in which one or more automobiles and one or more trucks were owned or kept at the same dwelling unit were reported for 12 percent of all dwelling units, and ranged from 5 percent in Louisiana to 30 percent in North Dakota, a situation which may be traceable to the difference in the types of agriculture practiced in the two States. As might be expected, the ownership of both automobiles and trucks in a household was concentrated in the unincorporated areas, where such ownership was reported for 22 percent of all households. In contrast, ownership of combinations of vehicles was reported for only 2 percent of the dwelling units in cities of 100,000 inhabitants or more.

Dwelling units for which the ownership of trucks only was reported included 7 percent of the six-State total, and ranged from 2 percent in Wisconsin to 16 percent in Arkansas. Among the various population groups the range was from 14 percent in the unincorporated areas to 1 percent in the incorporated places of 100,000 inhabitants and more.

Some of the differences in the chacteristics of motor-vehicle ownership among the six States may be observed more readily in figure 3. By far the largest percentage of families owning no vehicles were found in the two Southern States of Arkansas and Louisiana, 37 and 45 percent, respectively. On the other hand, the lowest percentage of families without motor vehicles was found in North Dakota where only 4 percent were reported. Ownership of one or more passenger cars and no trucks ranged from about 41 percent each in Arkansas and Louisiana to 65 percent each

Table 3.—Classification of occupied dwelling units by population groups, according to nature of motor-vehicle distribution; [1] summer of 1951, six States [2]

Classification	All population groups	Population group					
		Unincorporated areas	Incorporated places with population of—				
			All places	Less than 5,000	5,000–24,999	25,000–99,999	100,000 and over
Distribution by Population Grouping							
Dwelling units having:	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
One auto only	46.9	37.9	52.7	53.8	53.7	60.9	45.5
Two or more autos only	6.6	6.2	6.8	5.9	6.3	7.0	7.9
Auto(s) and truck(s) [3]	11.9	21.8	5.6	7.6	7.5	5.7	2.2
Truck(s) only [3]	7.1	14.1	2.6	4.9	2.6	1.6	1.3
All dwelling units having vehicles	72.5	80.0	67.7	72.2	70.1	75.2	56.9
Dwelling units having no autos or trucks	27.5	20.0	32.3	27.8	29.9	24.8	43.1
All occupied dwelling units	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Distribution by Dwelling-Unit Classification							
Dwelling units having:							
One auto only	100.0	31.6	68.4	19.4	16.2	15.2	17.6
Two or more autos only	100.0	37.0	63.0	15.1	13.6	12.6	21.7
Auto(s) and truck(s) [3]	100.0	71.5	28.5	10.6	8.9	5.6	3.4
Truck(s) only [3]	100.0	77.4	22.6	11.7	5.1	2.6	3.2
All dwelling units having vehicles	100.0	43.2	56.8	16.8	13.6	12.2	14.2
Dwelling units having no autos or trucks	100.0	28.5	71.5	17.1	15.3	10.6	28.5
All occupied dwelling units	100.0	39.2	60.8	16.9	14.1	11.7	18.1
Number of occupied dwelling units	*3,355,023*	*1,313,582*	*2,041,441*	*565,820*	*473,281*	*393,294*	*609,046*

[1] For the most part, the vehicles are owned in the places where the respondents reside. In some instances, however, they are garaged and/or owned elsewhere and operated regularly by a person or persons living at the dwelling unit.
[2] Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin.
[3] Including tractor-trucks but not semitrailers or trailers.

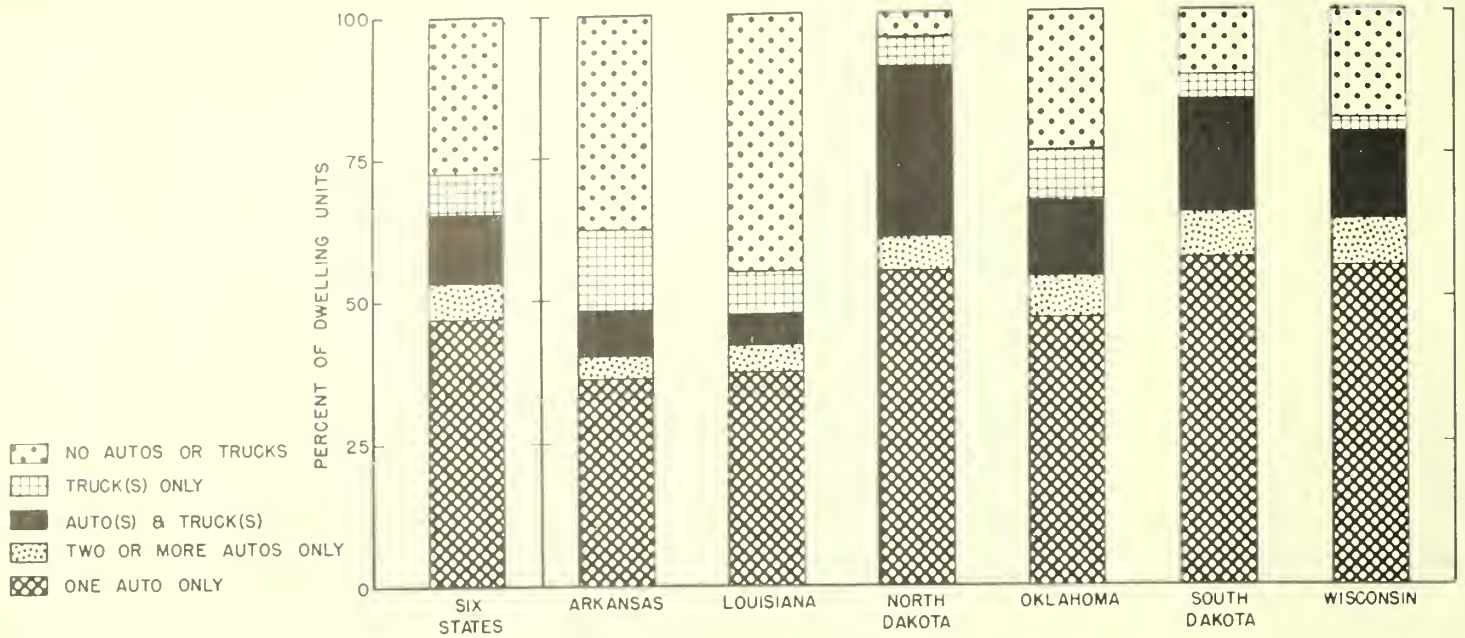

Figure 3.—Occupied dwelling units in each State classified by number of vehicles operated by residents; summer of 1951, six States.

in South Dakota and Wisconsin. The incidence of truck ownership either with or without automobiles was by far the greatest in North Dakota, where 35 percent of the dwelling units fell into this class.

Motor Vehicles on Farms

It is a common assumption that the average American farmer today is almost completely dependent upon motor vehicles to provide transportation for himself and his family, and to carry his products to market and his supplies to the farm. It has been generally assumed that in farming areas there is usually at least one motor vehicle owned at each operating farm unit, and that both automobiles and trucks will be found at many. The preliminary findings of the motor-vehicle-use studies not only substantiate this assumption, but also provide some very enlightening supplemental information.

Incidentally, if it appears that this report devotes considerable attention to the farmer and his use of motor vehicles, it should be remembered that the six States reported on are predominantly agricultural.

An indication of the reliability and stability of the motor-vehicle-use study samples is shown in table 4. This table indicates reasonable similarity in the distribution of farms according to acreage between the information obtained in the 1950 Census of Agriculture and that obtained in the motor-vehicle-use studies. As might be expected, the best comparison was found in Louisiana where the largest sample was taken. It will be noted that in four of the six States the percentage of farms containing less than 10 acres is greater in the motor-vehicle-use study totals than in the Census totals. This undoubtedly arises from differences in the definitions of farms used in the two investigations. In some States the motor-vehicle-use study interviewers classified as farms many small homesteads which were not so classified by the census enumerators. Other minor differences in definition are known to exist which would particularly affect the distribution of farms of less than 10 acres as compared with the other groups. One of these differences, resulting in a high percentage of farms of less than 10 acres, was the classification in the Wisconsin study of all dwelling units in the unincorporated areas as farms.

Figure 4 indicates, for the six States combined, the percentage distribution of motor-vehicle ownership by farm-acreage groupings. Over 84 percent of all families living on farms owned automobiles or trucks or both. Among the various acreage groups the range was from 52 percent on the farms of 10–29 acres to over 95 percent on the four farm-acreage groups of 180 acres or more. For farms of all sizes combined, the range among the samples obtained in the six States was from about 65 percent in Arkansas and Louisiana to 100 percent in North Dakota.

For all farms it may be noted that ownership of automobiles only was reported for 43 percent, with ownership of one or more automobiles together with one or more trucks accounting for an additional 26 percent. Ownership of trucks only was reported for 15 percent.

Table 4.—Comparisons of distributions of farms according to acreage as determined from 1950 Census of Agriculture and as reported in motor-vehicle-use studies; [1] summer of 1951, six States

Size group	Percentage distribution of farms according to acreage													
	Six States		Arkansas		Louisiana		North Dakota		Oklahoma		South Dakota		Wisconsin	
	Census	Study	Census	Study	Census	Study	Census	Study	Census	Study	Census	Study	Census	Study
Acres	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Less than 10	6.3	11.4	9.0	13.4	12.2	15.4	0.9	(2)	5.5	8.8	2.1	1.8	3.5	14.8
10–29	13.4	10.3	21.7	24.1	33.7	28.0	.9	(2)	7.3	4.9	1.8	(2)	4.1	4.1
30–49	10.9	8.8	17.5	15.8	21.0	21.2	.7	(2)	8.1	6.7	1.4	.5	6.6	5.4
50–99	17.1	19.3	22.0	21.9	16.3	19.0	1.4	(2)	15.8	18.2	3.3	2.7	25.0	26.8
100–179	20.9	23.3	16.7	14.1	8.4	8.6	8.2	4.7	25.6	34.7	15.7	29.9	37.5	30.5
180–259	8.8	8.3	5.9	5.5	2.9	3.2	5.0	2.7	11.0	7.3	12.0	9.5	14.5	12.9
260–499	12.7	9.8	4.8	2.8	2.9	3.2	35.7	41.6	17.3	14.3	32.9	22.2	7.8	3.9
500–999	6.6	5.8	1.7	1.7	1.5	1.2	33.8	36.9	6.5	3.6	17.6	17.6	.9	1.1
1,000 and over	3.3	3.0	.7	.7	1.1	.2	13.4	14.1	2.9	1.5	13.2	15.8	.1	.5
All farms	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Motor-vehicle-use studies exclude those few farms where acreage was not reported. [2] None reported.

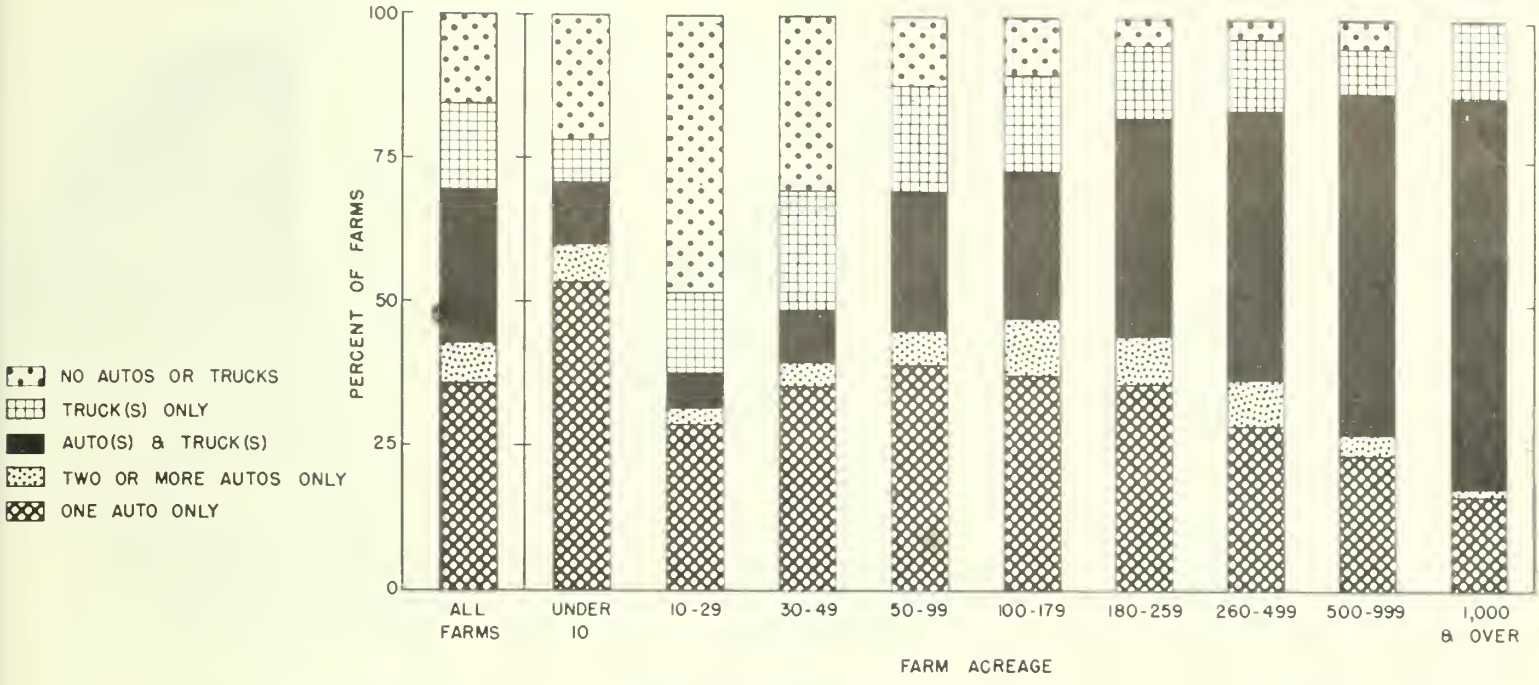

Figure 4.—Occupied farm dwelling units in each farm-acreage group classified by number of vehicles operated by residents: summer of 1951, six States.

Families living on farms of less than 10 acres had the highest percentage of one-automobile ownership, 53 percent. Perhaps one reason for the large number of one-automobile owners in this acreage group is that many of the people living on these small homesteads have other employment by which they obtain or supplement their income. Most of these "farms" probably do not produce sufficiently to warrant ownership of trucks.

Families living on farms of 10–29 acres had the highest percentage of non-ownership of vehicles, 48 percent, while those living on farms in the 30–49 acre group reported the next highest incidence of non-ownership with 30 percent. These figures are both much higher than the comparable 16 percent for all farm families. The farms of 10–29 acres and 30–49 acres probably include many subsistence-farming operations which produce only small quantities of products for sale. The Census data shown in table 4 clearly indicate the concentration of these smaller farms in Arkansas and Louisiana.

Less than 7 percent of farms in all acreage groups reported two or more automobiles only, with a range from 2 and 3 percent in Louisiana and Arkansas, respectively, to 11 percent in Wisconsin. Among the individual acreage groups, the range was from 2 percent in the case of the farms of 1,000 or more acres to 10 percent in the case of farms containing 100–179 acres.

Of all families living on farms of all sizes, 27 percent owned one or more automobiles and one or more trucks. Among individual acreage groups the range was from 6 percent in the 10–29 acreage group to 68 percent in the 1,000 or more acreage group. The percentage of farms having both automobiles and trucks rises rapidly as the acreage becomes larger. Arkansas and Louisiana reported the lowest percentage of families owning both cars and trucks, 8 and 9 percent, respectively, while the range in the other States was from 23 percent in Oklahoma to 58 percent in North Dakota.

In the six States, 15 percent of farm families owned trucks only, with a range from 7 percent on the farms of less than 10 acres to 21 percent on the farms of 30–49 acres. Many families residing on farms use a truck for all their travel needs. In three States, over 20 percent of all farms of all sizes had trucks only; the other three States reported less than 10 percent in this category.

Age of Automobiles in Use

In 1941, according to the Automobile Manufacturers Association, the average automobile in use in the United States was only 5.5 years old.[1] In 1951, according to this source, the average passenger car in use was 7.1 years old. The same source also indicates that the average age of motor vehicles at time of scrappage has increased fairly steadily from 6.5 years in 1925 to 13.5 years in 1950.[1] Since 1936 the average age at scrappage has increased at an annual rate of about one-third year. In 1941, for example, it was 10.2 years, and was probably about 13.8 years in 1951.

Better-built vehicles, better roads, wartime cessation of civilian motor-vehicle production, and improved economic conditions that have enabled more relatively low-income families to own automobiles are undoubtedly among the factors that have contributed to the increase in the average ages of vehicles now in service.

Table 5 indicates the distribution of automobiles in use classified by year models as reported by the motor-vehicle-use studies conducted in Arkansas, Louisiana, Oklahoma, and Wisconsin. Table 5 also compares the results of the motor-vehicle-use studies with the vehicle registrations for the same four States,[2] and the vehicle registrations for the United States.[1] The three distributions are closely parallel.

[1] *Automobile Facts and Figures*, 32nd ed., 1952, pp. 17, 19. Automobile Manufacturers Association, Detroit, Mich.

[2] *Automotive Industries*, 34th annual statistical issue, vol. 106, No. 6, March 15, 1952, pp. 108–115.

Table 5.—Passenger cars in use, classified by year model: motor-vehicle-use studies compared with vehicle registrations; summer of 1951, four States [1]

Year model	Motor-vehicle-use studies, summer of 1951, four States [1]	Vehicle registrations, July 1, 1951 [2]	
		Four States	United States
	Pct.	*Pct.*	*Pct.*
Postwar:			
1951	8.4	6.7	7.4
1950	15.8	15.2	15.9
1949	12.1	12.7	12.9
1948	7.2	7.7	7.7
1947	7.0	7.4	7.8
1946	4.6	5.1	5.1
Subtotal	55.1	54.8	56.8
War period, 1942–45	2.7	2.6	2.5
Prewar:			
1941	10.2	9.9	9.8
1940	7.8	7.3	7.3
1939	4.9	5.4	5.2
1938	2.9	3.6	3.2
1937	5.7	5.7	5.5
1936	4.2	10.5	3.9
1935 or older	5.7		5.6
Subtotal	41.4	42.4	40.4
Not reported [3]	.8	.2	.3
All year models	100.0	100.0	100.0
Number of vehicles, all year models	*2,172,461*	*2,334,755*	*38,516,000*

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.
[2] For sources, see text footnotes 1 and 2, this page.
[3] This table differs slightly from figure 5 in that it includes those vehicles for which the year model was not reported.

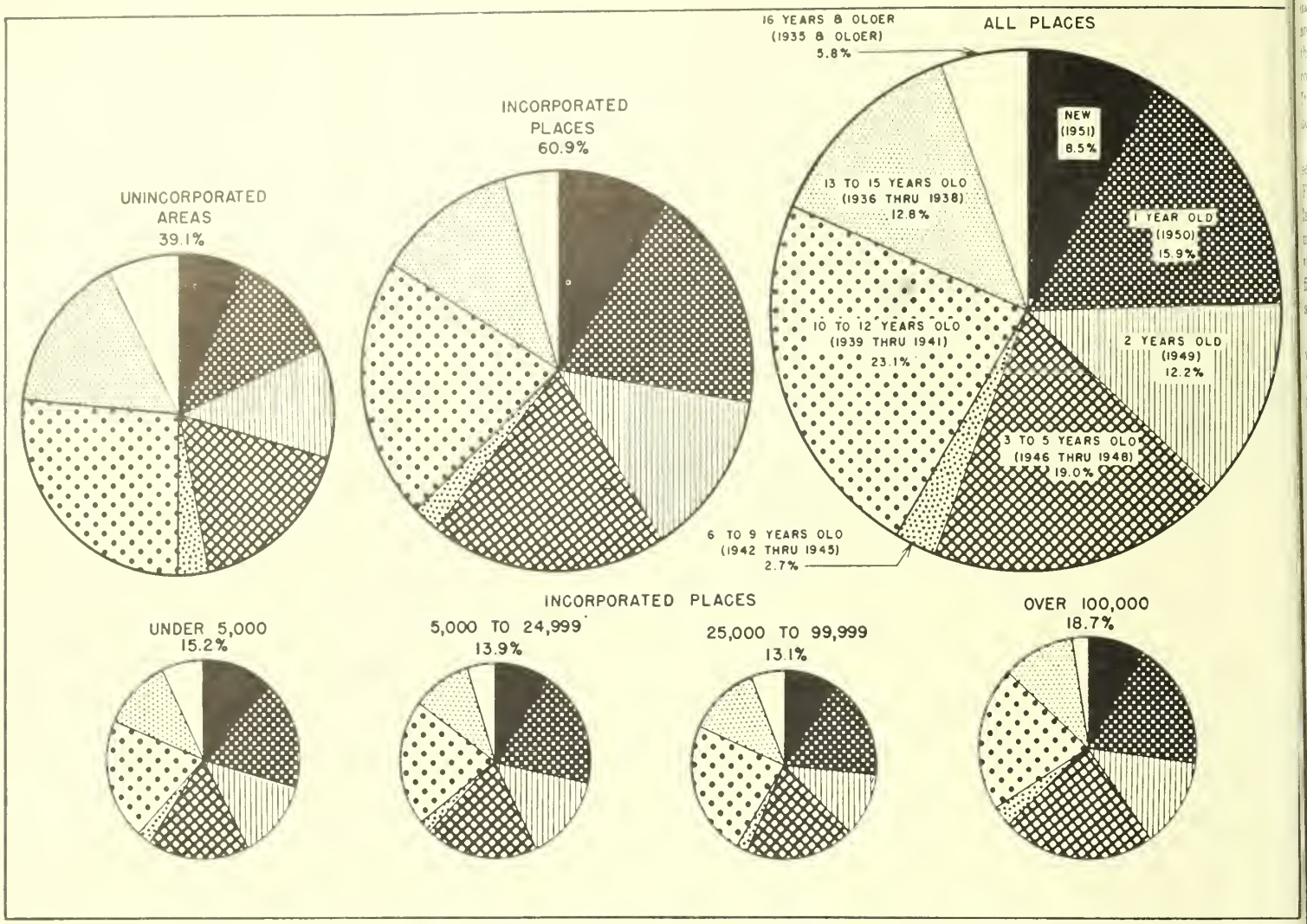
ALL PLACES
16 YEARS & OLDER (1935 & OLDER) 5.8%
NEW (1951) 8.5%
1 YEAR OLD (1950) 15.9%
2 YEARS OLD (1949) 12.2%
3 TO 5 YEARS OLD (1946 THRU 1948) 19.0%
6 TO 9 YEARS OLD (1942 THRU 1945) 2.7%
10 TO 12 YEARS OLD (1939 THRU 1941) 23.1%
13 TO 15 YEARS OLD (1936 THRU 1938) 12.8%
INCORPORATED PLACES 60.9%
UNINCORPORATED AREAS 39.1%
INCORPORATED PLACES
UNDER 5,000 15.2%
5,000 TO 24,999 13.9%
25,000 TO 99,999 13.1%
OVER 100,000 18.7%

Figure 5.—Passenger cars in use, classified by year model and by residence of principal operator; summer of 1951, four States.

Population-group characteristics

Figure 5 indicates the distribution of passenger cars in use by age and by population group of residence of the operators. The information shown is for the four States for which data are presented in table 5.

Slightly less than 61 percent of the automobiles reported upon were owned in incorporated places; the remaining 39 percent were owned in unincorporated areas. About 55 percent of all vehicles were postwar models, less than 3 percent were manufactured during World War II, and nearly 42 percent were manufactured before 1942.

The ownership of the newer vehicles—those produced since 1945—appeared to be more heavily concentrated in the incorporated places, especially in the cities having populations of 100,000 or more, where 64 percent of all vehicles were postwar models. Among the four States, the postwar-model automobiles found in all population groups ranged from 49 percent in Oklahoma to 60 in Arkansas.

For all population groups combined in all four States, 8 percent of the passenger cars reported were 1951 models. Among the individual States, the range was narrow, from 7 percent in Oklahoma to 10 percent in Arkansas. In considering the 1951 cars—the new cars at the time of the studies—it should be borne in mind that they had only been in production for a half-year, since the studies were conducted in the summer of 1951.

Almost 16 percent of all automobiles owned in all population groups combined were 1950 models, which were 1 year old at the time of this study; there were more of these than any other year model covered by this report. In Louisiana 19 percent of all automobiles reported were 1950 models, the highest percentage reported by any one of the four States.

Automobiles that were 2 years old—1949 models—accounted for 12 percent of all cars of all ages reported. There was relatively little variation in the ownership of these automobiles between the unincorporated areas and the incorporated places.

Passenger cars 3 to 5 years old—1946–48 models—accounted for only 19 percent of all vehicles reported. Although 1947 and 1948 models were found to represent a greater proportion of all vehicles owned in incorporated places than in unincorporated areas, the opposite was true for the 1946 models.

The ownership of vehicles produced during the war period, 1942–45, and before the war (prior to 1942) was found to be greater in the unincorporated areas. Fifty percent of all vehicles owned in unincorporated areas in the four States combined were prewar models, while an additional 3 percent were manufactured during the war. The corresponding percentages for incorporated places were 36 and 2, respectively. Among the individual States, Wisconsin reported the lowest percentage of war and prewar automobiles in the unincorporated areas, with 48 percent so shown. The highest percentage was 62 reported for Oklahoma.

The finding that older automobiles predominate among passenger cars owned in unincorporated areas is in line with the findings of earlier studies, such as the motor-vehicle-allocation and road-use studies. When these surveys were made in the middle and late 1930's it was believed that this finding was largely the result of the combined effects of the depression and drought that had been plaguing widespread portions of the rural areas of the nation. Now, however, it appears that the phenomenon is a continuing one, persisting in good times as well as bad.

Occupational-group characteristics

The occupational classification adopted for the presentation of motor-vehicle-use-study

data is essentially that adopted for the urban-area origin and destination studies made by the State highway departments with the cooperation of the Bureau of Public Roads. It is based upon a classification originally developed by the Bureau of the Census.

Although most of the classifications used are self-explanatory, some require clarification. For example, "craftsmen, foremen, and skilled laborers" include such occupations as boilermakers, carpenters, cement finishers, electricians, inspectors, locomotive engineers and firemen, mechanics, structural steel workers, and tailors. The terms "operatives" and "semiskilled workers" include apprentices, vehicle operators (such as bus and truck drivers), operators of stationary machines, miners, railroad brakemen, dockhands, seamstresses, and other workers from whom similar levels of training or skill are required. "Protective-service workers" include personnel in the armed forces, police, fire fighters, watchmen, and the like.

Although not an occupational group in the usual sense, the "housewife" category was set up separately because of the large number of instances in which it was found that the housewife was the owner, and often the sole operator, of a motor vehicle.

Figure 6 indicates the distribution of passenger cars by occupation of the principal operator and by age of vehicle. In nearly all cases reported the owner and principal operator were the same person. Consequently, the term owner is used in these discussions. These data are for the States of Arkansas, Louisiana, Oklahoma and Wisconsin.

It may be noted in figure 6, for the four States combined, that farmers and farm managers owned 20.2 percent of all cars of all year models. The next largest group of car owners was the operatives, semiskilled and unskilled workers, and laborers, with 20.0 percent of the vehicles so reported. Among the individual States there was little variation from this average. Craftsmen and foremen owned 17 percent of all the cars, with a range in the four States from 15 percent in Oklahoma to 20 percent in Louisiana. Store and office clerks owned 10 percent of all cars of all occupational groups, with a range among the four States from 6 percent in Wisconsin to 16 percent in Oklahoma.

As has been demonstrated in past studies, two factors, income and use requirements, seem to have an important bearing on the relative ages of cars owned by workers in various occupational groups. Thus, between 75 and 80 percent of all cars reported driven by traveling salesmen and agents, professionals and semiprofessionals, and proprietors, managers, and officials were postwar models. On the other hand, only 40 to 50 percent of the cars owned by farmers and farm managers, operatives, semiskilled and unskilled workers and laborers, and retired persons had been manufactured since the end of World War II.

Three of the occupational groupings are worthy of comment even though the total car ownership in two of them is relatively unimportant. These are the housewives, others, and occupation not reported classifications. The housewife group reported a considerably higher-than-average percentage of newer cars owned, while both of the other groups reported a much lower-than-average percentage of such vehicles. The explanation of these observed characteristics may be that the housewife group includes a relatively high representation of multi-car families, which are generally found in the higher-income brackets and so can afford to own newer automobiles. The other two groups probably include many students, itinerant workers, and persons of no regular occupation, all of whom generally have low incomes and so cannot afford to own the newer cars.

CHARACTERISTICS OF MOTOR-VEHICLE OPERATORS

Although at the present time the laws of all States and the District of Columbia require

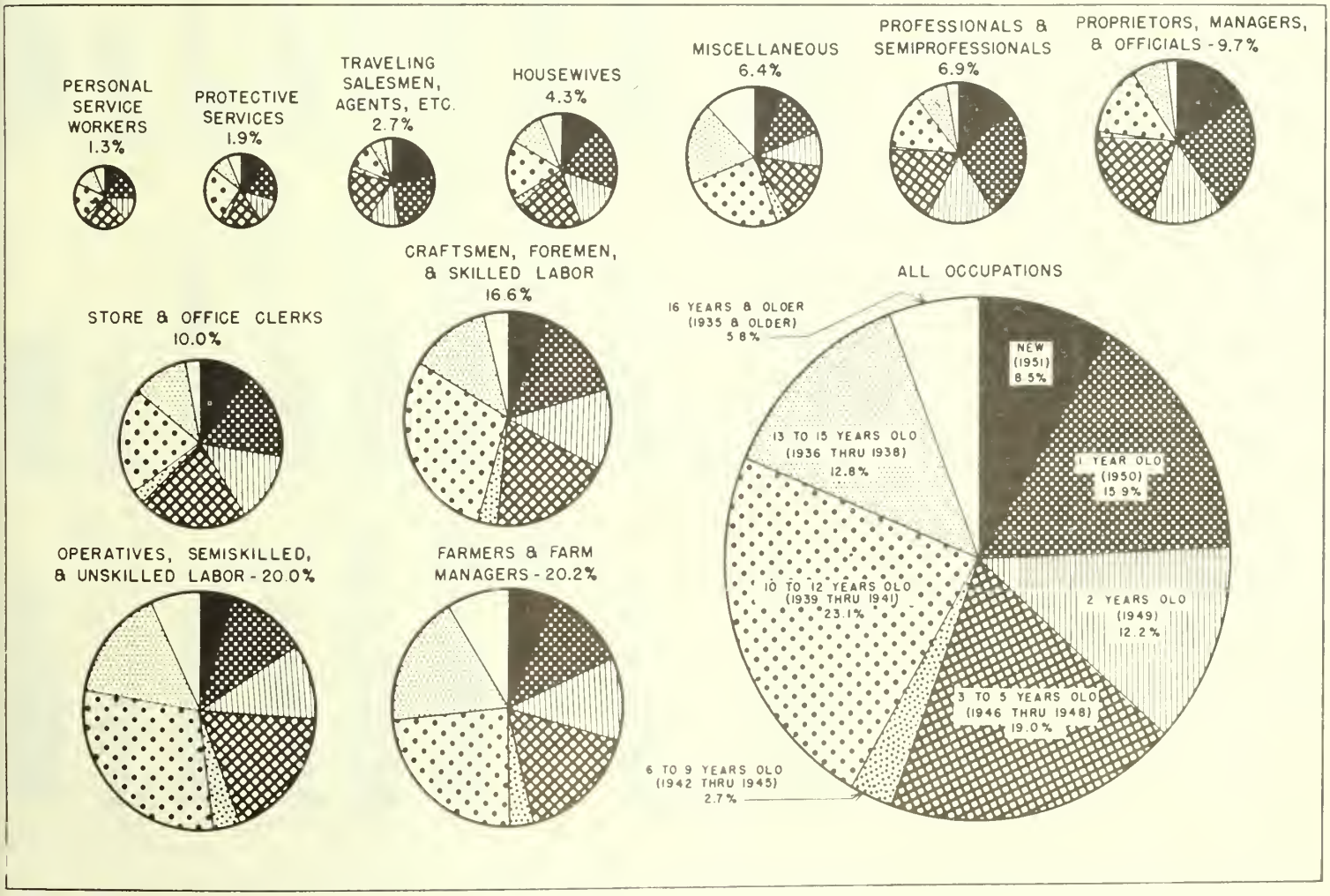
PERSONAL SERVICE WORKERS 1.3%
PROTECTIVE SERVICES 1.9%
TRAVELING SALESMEN, AGENTS, ETC. 2.7%
HOUSEWIVES 4.3%
MISCELLANEOUS 6.4%
PROFESSIONALS & SEMIPROFESSIONALS 6.9%
PROPRIETORS, MANAGERS, & OFFICIALS - 9.7%
CRAFTSMEN, FOREMEN, & SKILLED LABOR 16.6%
STORE & OFFICE CLERKS 10.0%
OPERATIVES, SEMISKILLED, & UNSKILLED LABOR - 20.0%
FARMERS & FARM MANAGERS - 20.2%
ALL OCCUPATIONS
16 YEARS & OLDER (1935 & OLDER) 5.8%
NEW (1951) 8.5%
1 YEAR OLD (1950) 15.9%
2 YEARS OLD (1949) 12.2%
3 TO 5 YEARS OLD (1946 THRU 1948) 19.0%
6 TO 9 YEARS OLD (1942 THRU 1945) 2.7%
10 TO 12 YEARS OLD (1939 THRU 1941) 23.1%
13 TO 15 YEARS OLD (1936 THRU 1938) 12.8%

Figure 6.—Passenger cars in use, classified by year model and by occupation of principal operator; summer of 1951, four States.

Table 6.—Proportion of total population licensed as motor-vehicle operators in each age-sex group for selected places of residence; summer of 1951, five States [1]

Age	Percentage of total population in each age-sex group licensed as motor-vehicle operators								
	All places			Unincorporated areas			Incorporated places		
	All persons	Female	Male	All persons	Female	Male	All persons	Female	Male
Years	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
14–15	8.1	4.1	11.8	11.1	6.0	15.3	4.9	2.3	7.6
16–20	46.1	26.1	66.1	49.9	26.8	67.8	42.1	25.6	63.9
21–29	65.1	46.5	86.7	67.2	47.4	87.8	63.7	45.9	85.9
30–39	68.6	50.2	87.9	70.5	52.4	88.7	67.3	48.7	87.4
40–49	63.6	43.5	84.4	66.1	42.8	87.8	61.8	43.9	81.9
50–59	52.4	27.3	79.4	56.4	27.9	83.5	49.6	27.0	76.1
60–69	37.9	14.6	61.0	39.6	10.3	63.1	36.7	17.2	59.2
70 and over	18.7	4.4	33.1	18.6	3.4	31.6	18.7	4.9	34.1
Age not reported	47.1	36.9	61.5	40.2	40.6	39.7	48.5	36.2	66.5
All licensed drivers	53.4	34.4	73.5	55.2	34.5	74.3	52.2	34.3	72.8

[1] Arkansas, Louisiana, North Dakota, Oklohama, and Wiseonsin.

that operators of motor vehicles be licensed, no such requirement was in effect in South Dakota when the limited motor-vehicle-use study was made in the summer of 1951. As a result, the data about motor-vehicle operators presented in this report are limited to five States: Arkansas, Louisiana, North Dakota, Oklahoma, and Wisconsin.

The data obtained in the five States indicate (table 6) that 53 percent of all persons 14 years old or older residing in those States then had permits to drive automobiles. The highest incidence of permit holders was 69 percent in the 30–39-year age group. These findings, and others discussed subsequently, compare very favorably with the findings of studies made recently in several States by the Automobile Manufacturers Association.[3]

According to the motor-vehicle-use studies, 67 percent of the licensed drivers residing in the five States were males while 33 percent were females.

As shown in table 6 and figure 7, 74 percent of all males over 13 years old living in those States were licensed drivers, while only 34 percent of all females over 13 years old were licensed. Among the individual age groupings the highest incidence of drivers, both men and women, was in the 30–39-year age group, the percentages for the sexes being 88 and 50, respectively. Actually, there was relatively little deviation from these percentage relations in the 21–29 or 40–49 age groups. The percentage of all males 50–59 years of age licensed to drive was only slightly smaller, 79 percent, but the corresponding percentage for females was only 27 percent. The relative percentages of both male and female drivers to corresponding age-group totals were much smaller under 21 and over 59 years of age than for any age group between those limits.

The 14–20-year age groups require special consideration because the percentages of persons within these groups who were licensed to drive automobiles were directly affected by the licensing laws of the individual States. In Arkansas, North Dakota, Oklahoma, and Wisconsin junior driver permits might be obtained at age 14 under certain conditions. Although junior permits were not issued in Louisiana, a regular driver's license might be obtained there at age 15. In North Dakota, Oklahoma, and Wisconsin the minimum age for a regular operator's license was 16; in Arkansas it was 18. Among the five States the percentage of all drivers under 20 years of age ranged from 8 percent in Arkansas and Wisconsin to 14 percent in North Dakota.

The unincorporated areas and the incorporated places reported about the same percentage of male and female drivers of total male and female population for all age groups combined as was reported for all population groups. With few exceptions, only minor variations may be noted among the various age groups. The percentage of drivers 14 and 15 years of age was more than twice as high in the unincorporated areas as in the incorporated places. In almost every age group the percentages of drivers, both male and female, were a little higher in the unincorporated areas than in the incorporated places.

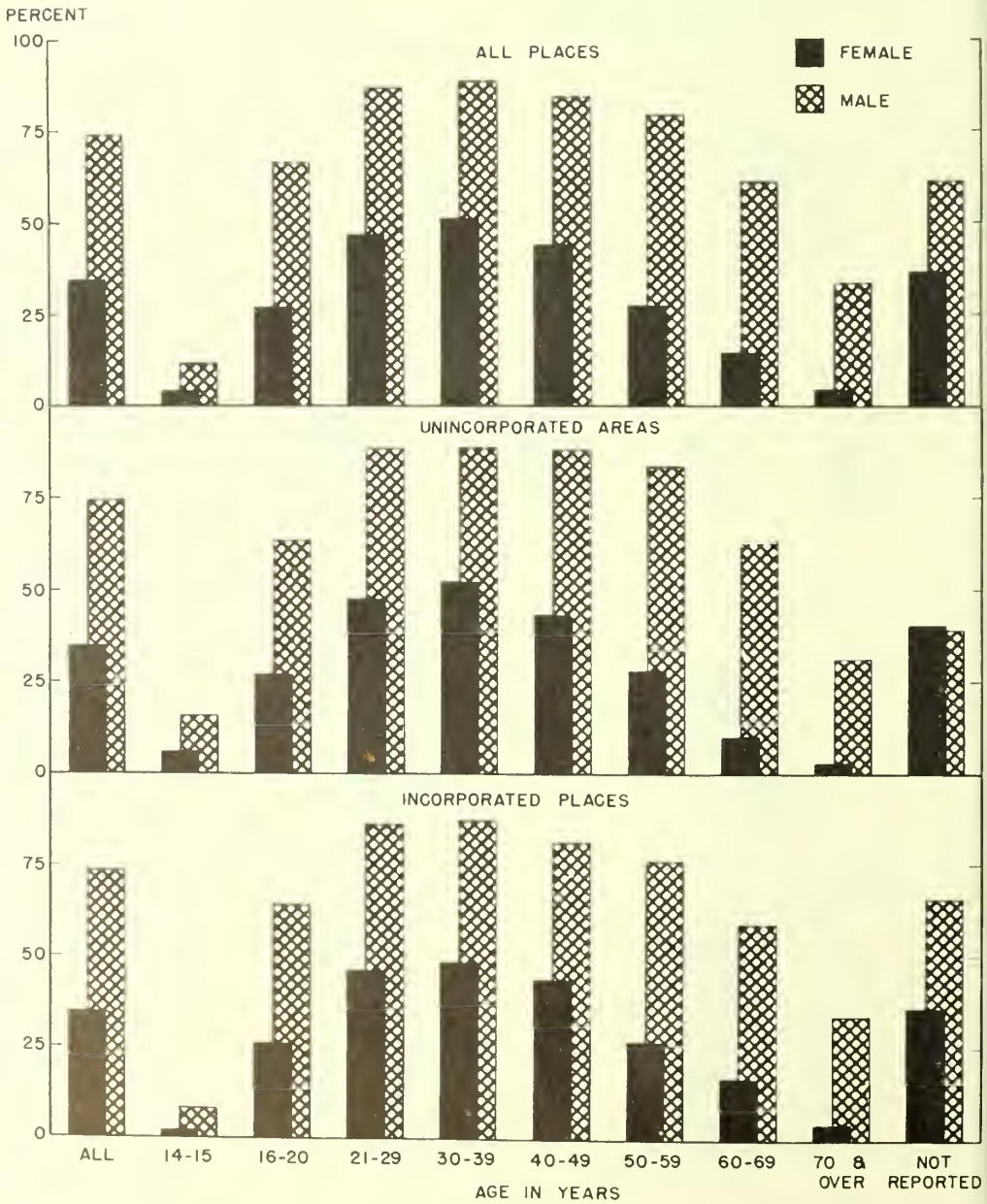

Figure 7.—Proportion of total population in each age-sex group licensed as motor-vehicle operators, for selected places of residence; summer of 1951, five States.

[3] For some results of these studies see *Automobile Facts and Figures*, 33rd ed., 1953, p. 36. Automobile Manufacturers Association, Detroit, Mich.

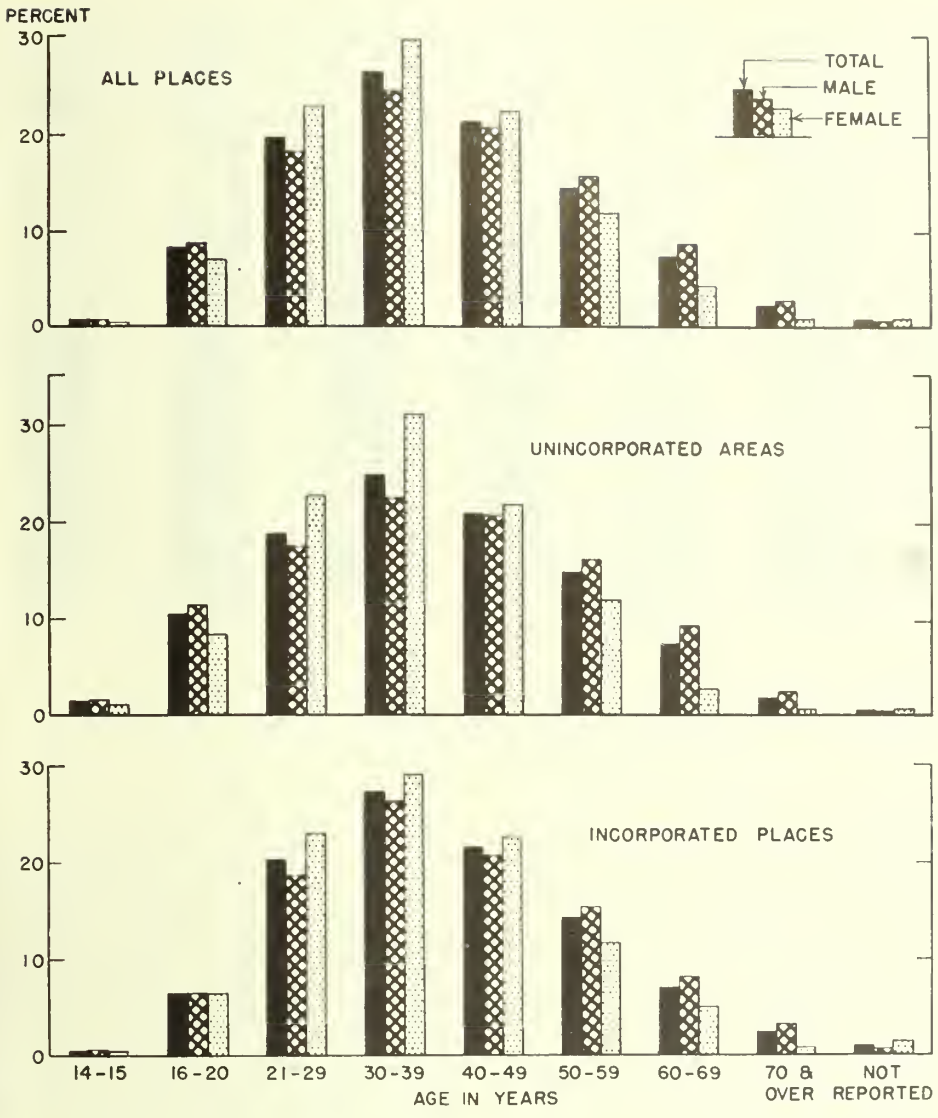

Figure 8.—Age distribution of licensed motor-vehicle operators for each sex group, for selected places of residence; summer of 1951, five States.

Some of the characteristics already mentioned are placed in a different perspective in figure 8. For all population groups combined in the five States, over 26 percent of all drivers of all ages were between 30 and 39 years of age. About 30 percent of all female drivers and 24 percent of all male drivers were in this age group. It may be noted also that 75 percent of all female drivers and 63 percent of all male drivers were between 21 and 49 years of age. In the 14–20-year age groups, the percentage of male and female drivers were reported as 9 and 7 percent, respectively.

The distribution of all drivers in the unincorporated areas reflects the influence of the importance of the motor vehicle to the rural areas. A considerably higher percentage of younger motor-vehicle operators was reported for the unincorporated areas than for the incorporated places: In the unincorporated areas 11 percent of all drivers were under 21 years of age as compared with 7 percent in all incorporated places. At the other extreme, the percentage of drivers over 60 years of age did not vary markedly between the unincorporated areas and the incorporated places.

TRAVEL TO AND FROM WORK

Most employed persons find it necessary to travel to and from their places of employment. thereby producing the rush-hour traffic congestion found in urban areas. Travel to and from work was the cause of more than 30 percent of the trips and over 20 percent of the mileage traveled by the average private automobile. Because of the importance of this segment of passenger travel, and its bearing on many aspects of the provision of highway facilities, it was given rather extensive consideration in the motor-vehicle-use studies.

Table 7 indicates, for the six States covered in this report, the total number of gainfully employed persons in each occupational group and the percentages traveling between their homes and their places of employment. (In general, in this report, walking is included among the modes of travel to and from work.) It will be noted that 68 percent of all workers in all occupational groups combined came into this category; the remaining 32 percent worked at their places of residence. As would be expected, only 9 percent of the farmers and

Table 7.—Percentages of gainfully-employed persons who travel to work,[1] classified by occupation; summer of 1951, six States [2]

Occupational group	Number of gainfully employed workers [3]	Percentage traveling to work [1]
Professionals and semiprofessionals	246,820	88.7
Proprietors, managers, and officials	284,667	81.0
Farmers and farm managers	1,004,119	9.2
Store and office clerks, salesmen, etc.	505,784	95.7
Traveling salesmen, agents, etc.	54,455	88.5
Craftsmen, foremen, skilled laborers, etc.	512,983	94.3
Operatives, semiskilled and unskilled workers, and laborers	894,809	86.9
Protective services	61,823	73.5
Personal-service workers	180,612	88.0
All occupations [3]	3,746,072	67.8

[1] Including walking.
[2] Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin.
[3] Excludes "occupation not reported" and those for which the method of travel was not reported.

farm managers traveled to work; a far lesser proportion than for any other group.

Modes of Travel to and from Work

Figure 9 and table 8 indicate the extent of use of various modes of travel by workers residing in each of the various population groups. As used here, the term public transportation includes all types of carriers, such as city transit lines, intercity bus lines, and railroads. However, transportation by truck or bus provided by the employer would be classified under other means, as would, for example, travel to and from work by motorcycle or bicycle, or in a truck owned by the worker.

For all population groups combined, 61 percent of the workers used passenger cars alone for home-to-work travel. Among the population groups, the range was from 46 percent in incorporated places having a population of 100,000 or over to 73 percent in unincorporated areas. The next most common means of travel was walking, with 16 percent of all persons walking to and from work. The smallest proportion of walkers, as one might expect, was in the unincorporated areas (8 percent); the largest was in incorporated places having a population under 5,000 (27 percent). Use of public transportation alone was the highest, 38 percent, in the places having a population of 100,000 or over and lowest, 2 percent, in places under 5,000. This is about what would be expected—the larger the place, the more important would be the public transportation system. Trips in which both public transportation and automobiles were used in combination amounted to less than 2 percent.

Relation Between Distance to Work and Mode of Travel

It is generally recognized that there is a definite relation between distance from home to work and mode of travel used in getting to and

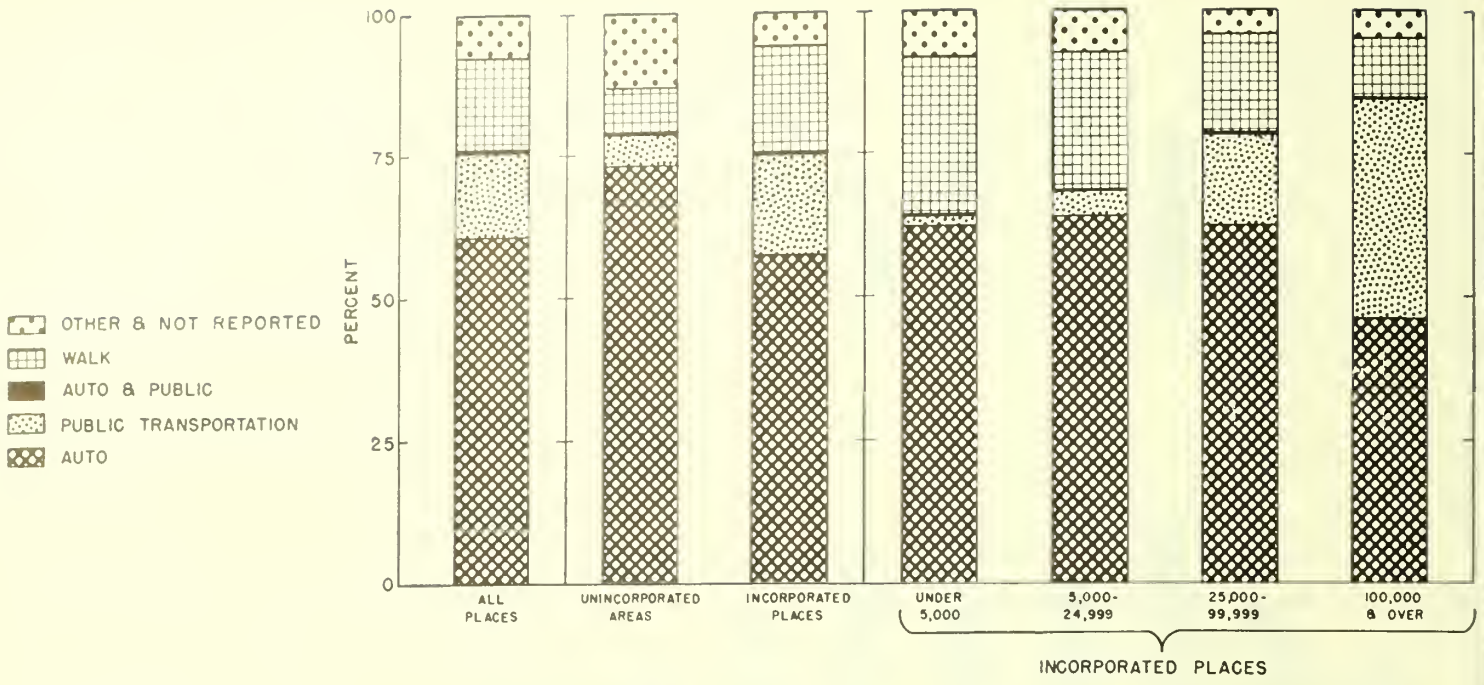

Figure 9.—Gainfully-employed workers in each population group using various modes of home-to-work travel; summer of 1951, six States.

Table 8.—Percentages of gainfully-employed workers in each population group using various modes of home-to-work travel; summer of 1951, six States [1]

Population group	Number of workers	Percentage of workers, by mode of travel						
		All modes of travel	Passenger cars	Public transportation	Passenger cars and public transportation	Walk	All other means	Method not reported
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Unincorporated areas	591,746	100.0	73.3	5.3	1.8	8.1	6.2	5.3
Incorporated places:								
Under 5,000	496,177	100.0	62.6	1.6	1.1	27.4	3.5	3.8
5,000–24,999	476,151	100.0	64.4	4.2	.9	24.2	2.1	4.2
25,000–99,999	446,554	100.0	62.8	15.7	.9	17.2	1.8	1.6
100,000 and over	715,907	100.0	46.4	38.4	2.2	9.9	1.3	1.8
Subtotal	2,134,789	100.0	57.6	17.5	1.4	18.7	2.0	2.8
All places	2,726,535	100.0	61.0	14.8	1.5	16.4	3.0	3.3

[1] Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin.

from work—a hypothesis supported by these motor-vehicle-use studies.

Table 9 presents the combined findings of the six State studies reported upon. For all population groups except incorporated places having populations under 5,000 inhabitants, more persons who lived less than a mile from their places of employment walked to work than traveled by any other means. However, when the distance to work was 1 mile or more, the percentage of workers traveling to and from their jobs entirely by automobile exceeded 68 percent in all cases except in cities having populations of 100,000 inhabitants or more, where the percentage was 40. Of all persons walking to work, 85 percent lived within 1 mile and more than 99 percent within 3 miles of their work.

Public transportation was relied upon more heavily by workers living in cities having 25,000 or more inhabitants than by those residing elsewhere. About 17 percent of workers living in places having populations of 25,000–100,000 traveled from home to work by public transportation for all or part of their trips; for cities of 100,000 or more inhabitants the comparable percentage was 41. However, in both classes of places it was the workers who lived from 1 to 5 miles from their jobs who relied most heavily upon public transportation in getting to and from work.

Distribution of Workers Using Automobiles

In the six States reported upon, about 65 percent of all persons using automobiles in going to and from their jobs traveled 5 miles or less one way on such trips (fig. 10). Conversely, only 8 percent traveled 20 miles or more each way between their homes and places of employment.

In all of the frequency distributions shown in figure 10, the great proportion of trips were in the shortest distance ranges, except for the unincorporated areas and the cities having populations in excess of 100,000. The significance of these relations becomes more apparent when they are considered along with the relations shown in figure 9. People who live in unincorporated areas and incorporated places having populations of less than 25,000 relied very little upon public transportation as a means of getting to and from work. Therefore, most residents of such areas who traveled to and from work either walked or else drove or rode in automobiles. In the medium-sized cities (25,000–100,000 inhabitants) public transportation was an important factor but, probably because of the relatively short distances involved, its effect seemed to be mainly to reduce the percentage of walkers rather than that of automobile users. Only in the larger cities (those having population of 100,000 or more) did public transportation become a major factor in transporting workers; here the percentage of workers using automobiles was at its lowest level. Even in these larger places, however, the percentage of workers traveling by automobile (46 percent) exceeded the percentage using mass transportation for the entire trip or in combination with auto travel (total of 41 percent).

In figure 10 the total number of automobile users represents nearly 75 percent of all workers living in unincorporated areas requiring home-to-work travel, about 62 to 64 percent of such workers living in incorporated places having populations of less than 100,000, and 46 percent of such workers living in larger places.

Workers living in unincorporated areas include both those living in suburban sections and those living in rather remote open-country areas. It was not unexpected, therefore, to find that 55 percent of these workers who go to and from work by automobile traveled more than 5 miles one way, or that 11 percent

of them traveled 25 miles or more one way.

Although a goodly number of workers living in incorporated places having less than 5,000 inhabitants worked elsewhere, as was evidenced by the more than 8 percent who traveled more than 25 miles by car to their jobs, nearly two-thirds of the automobile users worked within 5 miles of their homes, and 40 percent within 1 mile of home. In the next larger class of places, only 3 percent of the workers traveled over 25 miles to their jobs, while 78 percent traveled less than 5 miles—indicating, it would seem, that these communities were much more self-contained than the smaller places.

Only 1.5 percent of automobile users living in places having populations of 100,000 or more traveled 25 miles or more to work. On the other hand, while 65 percent reported traveling less than 5 miles to work, only 20 percent traveled less than 2 miles. Evidences of the effects of larger area and increased use of public transportation for short trips appear to be present.

Relation of Occupation and Mode of Travel to Work

Two factors might be expected to have an important bearing upon whether or not workers in various occupational groups would be likely to use automobiles in traveling between home and work: The need for a car on the job, and the nature of the individual occupational group.

Table 10 indicates the relative distribution of modes of travel used in going to and from their jobs by workers in the various occupational groups, as shown by the findings of motor-vehicle-use studies made in the six States. In all but one of the five classes of places, traveling salesmen and agents used automobiles more than any other occupational group. The one exception was the proprietor, manager, and official group in the cities having a population of 100,000 or more. Most traveling salesmen and agents, of course, find it necessary to use automobiles in connection with their work.

In the four groups of places where traveling salesmen and agents led in the percentage of workers using automobiles, the proprietor, manager, and official group was second in two, while the craftsmen, foremen, and skilled laborers group was second in one and the protective-service group was second in the other.

The personal-service worker group was lowest in percentage of car use in all population groups.

Table 9.—Distribution of gainfully-employed workers traveling from home to work, classified by mode of travel within each distance group for all places of residence; summer of 1951, six States[1]

Population group and one-way distance	Number of workers[2]	Distribution of workers, by mode of travel					
		All modes of travel	Passenger car	Public transportation	Passenger car and public transportation	Walk	All other means and not reported
Miles		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
All places:							
0.1–0.9	690, 176	100.0	42.9	3.3	.7	50.5	2.6
1.0–1.9	425, 162	100.0	66.5	18.3	1.3	12.0	1.9
2.0–2.9	292, 710	100.0	65.4	28.5	1.1	2.5	2.5
3.0–4.9	371, 195	100.0	65.7	28.6	2.2	.4	3.1
5.0–9.9	340, 541	100.0	77.0	18.4	1.6	.4	2.6
10.0–19.9	204, 492	100.0	84.1	9.5	2.3	------	4.1
20 and over	136, 358	100.0	84.5	6.1	2.7	------	6.7
Not reported	265, 901	100.0	37.7	9.1	1.4	13.9	37.9
All distances	2, 726, 535	100.0	61.0	14.8	1.5	16.4	6.3
Unincorporated areas:							
0.1–0.9	65, 639	100.0	41.9	.8	1.8	52.2	3.3
1.0–1.9	42, 154	100.0	72.7	1.6	5.9	13.9	5.9
2.0–2.9	55, 725	100.0	85.3	3.7	------	5.6	5.4
3.0–4.9	85, 412	100.0	89.5	4.3	.5	.3	5.4
5.0–9.9	117, 089	100.0	82.8	10.8	1.9	.2	4.3
10.0–19.9	80, 398	100.0	81.1	8.0	3.6	------	7.3
20 and over	71, 177	100.0	84.7	5.8	------	------	9.5
Not reported	74, 152	100.0	39.4	1.7	2.3	6.1	50.5
All distances	591, 746	100.0	73.3	5.3	1.8	8.1	11.5
All incorporated places:							
0.1–0.9	624, 537	100.0	43.0	3.6	.6	50.3	2.5
1.0–1.9	383, 008	100.0	65.8	20.1	.9	11.8	1.4
2.0–2.9	236, 985	100.0	60.8	34.4	1.4	1.7	1.7
3.0–4.9	285, 783	100.0	58.6	35.9	2.8	.4	2.3
5.0–9.9	223, 452	100.0	74.0	22.4	1.4	.5	1.7
10.0–19.9	124, 094	100.0	86.0	10.5	1.5	------	2.0
20 and over	65, 181	100.0	84.2	6.5	5.7	------	3.6
Not reported	101, 749	100.0	37.0	12.0	1.1	16.9	33.0
All distances	2, 134, 789	100.0	57.6	17.5	1.4	18.7	4.8
Incorporated places under 5,000:							
0.1–0.9	234, 007	100.0	48.4	------	.4	47.5	3.7
1.0–1.9	49, 519	100.0	77.8	------	------	20.7	1.5
2.0–2.9	11, 064	100.0	86.5	1.0	4.4	3.0	5.1
3.0–4.9	21, 634	100.0	82.9	5.1	5.0	------	7.0
5.0–9.9	44, 364	100.0	87.2	6.5	1.6	1.4	3.3
10.0–19.9	42, 062	100.0	90.3	2.0	3.6	------	4.1
20 and over	33, 398	100.0	89.9	3.9	1.8	------	4.4
Not reported	60, 129	100.0	40.6	3.2	.3	22.5	33.4
All distances	496, 177	100.0	62.6	1.6	1.1	27.4	7.3
Incorporated places of 5,000–24,999:							
0.1–0.9	188, 558	100.0	47.2	1.7	.5	48.4	2.2
1.0–1.9	115, 541	100.0	80.3	6.1	1.8	10.3	1.5
2.0–2.9	31, 351	100.0	80.5	10.8	1.7	2.0	5.0
3.0–4.9	19, 661	100.0	85.7	10.6	------	3.7	------
5.0–9.9	21, 754	100.0	90.0	4.2	2.4	------	3.4
10.0–19.9	30, 776	100.0	95.2	4.0	------	------	.8
20 and over	14, 593	100.0	90.0	8.4	------	------	1.6
Not reported	53, 917	100.0	38.5	1.5	.4	19.7	39.9
All distances	476, 151	100.0	64.4	4.2	.9	24.2	6.3
Incorporated places of 25,000–99,999:							
0.1–0.9	106, 180	100.0	38.8	5.2	1.0	53.0	2.0
1.0–1.9	119, 120	100.0	68.2	17.9	.3	12.0	1.6
2.0–2.9	89, 032	100.0	71.0	26.2	.5	1.5	.8
3.0–4.9	54, 456	100.0	73.5	23.1	------	.8	2.6
5.0–9.9	34, 868	100.0	87.3	8.6	1.0	1.3	1.8
10.0–19.9	12, 517	100.0	87.2	6.7	3.4	-	2.7
20 and over	8, 133	100.0	71.2	9.1	15.6	------	4.1
Not reported	22, 248	100.0	34.5	13.1	------	17.9	34.5
All distances	446, 554	100.0	62.8	15.7	.9	17.2	3.4
Incorporated places of 100,000 and over:							
0.1–0.9	95, 792	100.0	26.0	14.2	1.1	58.0	.7
1.0–1.9	98, 828	100.0	40.1	49.1	.8	8.7	1.3
2.0–2.9	105, 538	100.0	43.5	51.8	1.7	1.7	1.3
3.0–4.9	190, 032	100.0	48.7	45.7	3.6	------	2.0
5.0–9.9	122, 466	100.0	62.5	35.2	1.2	.1	1.0
10.0–19.9	38, 739	100.0	73.6	25.9	------	------	.5
20 and over	9, 057	100.0	65.7	10.6	19.9	------	3.8
Not reported	55, 455	100.0	32.7	31.2	3.0	7.8	25.3
All distances	715, 907	100.0	46.4	38.4	2.2	9.9	3.1

[1] Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin.
[2] These numbers differ slightly from those shown in table 10 because this table excludes 36,096 persons for whom occupation was reported but both the method of transportation and the mileage to work were not reported.

DISTRIBUTION OF AUTOMOBILE TRAVEL

Perhaps the most pressing need for the information obtainable in the motor-vehicle-use studies is for data concerning the distribution of motor-vehicle travel by road systems, by type of area or size of place where the travel is performed, and by place of residence of the vehicle user. A consistent tendency for respondents to interviews to under-report travel has been observed in other studies of this nature. Consequently, it can be expected that a tendency for some under-reporting of trip data will occur in these studies, with the result that the total vehicle-miles traveled may be somewhat understated. It is believed, however, that the percentage distribution of travel will be reasonably representative of the characteristics of the total amount of travel performed.

Since the travel information available for this preliminary report includes data from only four States in the central region of the United States, it quite probably does not adequately represent the United States as a whole, particularly in the rural and urban travel distribution, and should be considered in that light. It does, however, give a picture of the type of information collected.

Distribution by Place of Travel

Rural roads.—The main rural roads (primary State highways) carried about one-half of the automobile travel of both rural and urban residents, as shown in figure 11 and table 11. About 49 percent of the total automobile travel performed by residents of un-

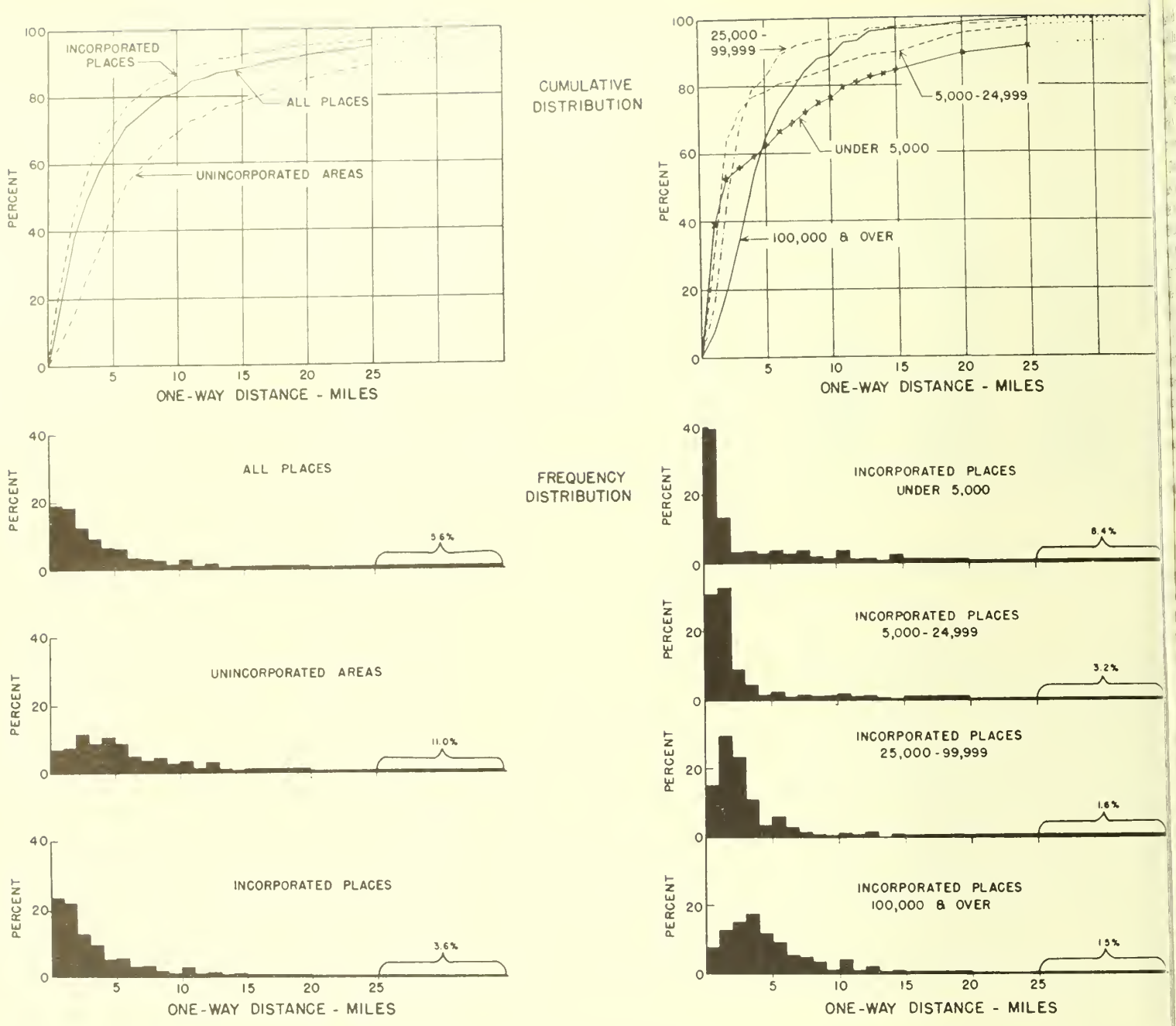

Figure 10.—Gainfully-employed workers using passenger cars for home-to-work travel, classified by distance to place of employment and by place of residence; summer of 1951, six States.

incorporated areas and 47 percent of the total travel performed by residents of incorporated places in these States occurred on the primary rural-road system within the State of residence. This percentage varied from a low of 33 percent for residents of cities having a population of 100,000 or more to a high value of 59 percent for those living in places with a population of less than 5,000.

Among the individual States the range in passenger-car travel between main rural roads and other rural roads varied somewhat, primarily because of differences in the proportionate extent of rural-road mileages in the primary highway systems of these States. This variation ranged from 40 percent for the State primary roads of Louisiana to 69 percent in Arkansas. The local rural-road travel varied in opposite order, from 7 percent in Arkansas to 31 percent in Louisiana. This, of course, may be explained by the fact that the primary rural-road mileage of Louisiana amounted to only 6 percent of the total rural-road mileage of the State, whereas 14 percent of the rural roads in Arkansas were on the State primary-road system.

Residents of the unincorporated areas used the local rural-road system of their States proportionately more than the urban residents: 32 percent of their automobile travel was on local roads as compared with 7 percent for residents of all incorporated places. This use of local roads by residents of incorporated places varied from a high of 13 percent for residents of the smaller places to 3 percent for those living in the larger cities.

City streets.—As would be expected, urban residents drove their cars on the streets in the incorporated places within the State of residence (36 percent of their travel) to a much larger extent than did the rural residents (14 percent). In the incorporated places, the range was from 21 percent for residents of cities having a population under 5,000 to 58 percent in the cities having a population of 100,000 or more.

In considering the travel performed in the incorporated places within the State by the residents of these States, it may be noted from figure 12 and table 12 that slightly over one-half of the passenger-car travel within these places resulted from trips confined entirely within cities. The percentage of city travel represented by trips entirely within cities

aried from 18 percent in the cities having a opulation of less than 5,000 to 75 percent n the cities having a population of 100,000 r more. It will be noted in table 12 that he variation from State to State was rela-ively small within each of the population roups.

Total travel.—Of all travel by passenger ars in these four States, 71 percent was in he unincorporated areas and 29 percent in he incorporated places, as shown in table 11 nd figure 11. Among the individual States, he range for travel in incorporated places vas from 24 percent for Arkansas to 30 per-ent for Louisiana and Oklahoma.

Out-of-State travel.—Approximately 8 per-ent of the travel performed by residents of hese four States was on roads and city streets n other States. Among individual States, here was very little variation in the percent-ge of out-of-State travel.

Occupational Distribution of Automobile Travel

Approximately one-half—ranging from 43 ercent to 53 percent—of the total passenger-ar travel of each occupational group was per-ormed on the primary highways within the tate of residence, as shown in figure 13 and able 13.

Farmers and farm managers reported 37 ercent of their driving on other rural roads within the State of residence—far more than he 9 to 19 percent for the various other ccupational groups.

The farmers and farm managers drove only 3 percent of their total in-State mileage on treets in incorporated places. As a result of heir low proportion of city driving and their elatively high proportion of driving on other ural roads, they had about the same propor-ion of travel on the primary highways as all ther occupational groups.

Three groups—professionals and semipro-essionals, proprietors, managers, and officials, nd store and office clerks and salesmen—per-ormed a greater proportion of travel outside f their State of residence than did the other ccupational groups, reporting out-of-State travel of about 11 percent. Farmers and farm managers and miscellaneous (including retired persons, other unclassified occupations, and occupations not reported), reported that less than 3 percent of all their travel was outside the State of residence. The other occupational groups showed a range from 5 to 9 percent.

Figure 14 shows the distribution of travel and the number of automobiles by occupational groupings for each place of residence of principal operator. For all population groups combined, five of the eleven occupational groups traveled a higher percentage of the total vehicle-miles than the corresponding proportion of automobiles owned by these people. For instance, the professional and semiprofessional group traveled 10 percent of all the vehicle-miles while operating only 7 percent of the cars.

The relative distributions among occupational groups for each of the four groups of

Table 10.—Distribution of gainfully employed workers traveling from home to work, classified by mode of travel within each occupation group for each place of residence; summer of 1951, six States [1]

Residence and occupational group	Number of workers [2]	Distribution of workers, by mode of travel					
		All modes of travel	Passenger car	Public transportation	Passenger car and public transportation	Walk	All other means and not reported
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
All places:							
Professionals and semiprofessionals	230,098	100.0	65.2	11.2	0.9	16.8	5.9
Proprietors, managers, officials [3]	334,062	100.0	73.7	3.2	.8	14.4	7.9
Store and office clerks, salesmen	492,892	100.0	59.3	18.6	2.0	17.4	2.7
Traveling salesmen, agents, etc	49,599	100.0	82.6	5.9	1.9	4.6	5.0
Craftsmen, foremen, skilled laborers	494,260	100.0	71.7	11.2	1.3	10.2	5.6
Operatives, workers, and laborers	805,015	100.0	59.3	14.7	1.5	16.8	7.7
Protective services	53,379	100.0	66.0	8.3	2.0	8.2	15.5
Personal-service workers	173,857	100.0	22.5	34.4	.6	32.0	10.5
Miscellaneous	8,516	100.0	74.9	5.7		15.8	3.6
Occupation not reported	120,953	100.0	15.4	32.1	3.1	21.2	28.2
All occupations	2,762,631	100.0	61.0	14.8	1.5	16.4	6.3
Unincorporated areas:							
Professionals and semiprofessionals	30,385	100.0	78.0	4.6		6.5	10.9
Proprietors, managers, officials [3]	95,435	100.0	67.2	.2	.6	17.7	14.3
Store and office clerks, salesmen	68,953	100.0	83.1	5.6	1.0	5.0	5.3
Traveling salesmen, agents, etc	3,457	100.0	90.5				9.5
Craftsmen, foremen, skilled laborers	106,953	100.0	85.7	2.9	.8	2.3	8.3
Operatives, workers, and laborers	224,008	100.0	75.5	1.2	3.0	6.4	13.9
Protective services	14,073	100.0	60.8	2.8		2.8	33.6
Personal-service workers	16,770	100.0	48.0	5.2		19.0	27.8
Miscellaneous	1,478	100.0	50.0			36.9	13.1
Occupation not reported	43,856	100.0	10.4	49.7	5.0	13.7	21.2
All occupations	605,368	100.0	73.3	5.3	1.8	8.1	11.5
All incorporated places:							
Professionals and semiprofessionals	199,713	100.0	63.3	12.3	1.0	18.3	5.1
Proprietors, managers, officials [3]	238,627	100.0	76.3	4.4	.9	13.1	5.3
Store and office clerks, salesmen	423,939	100.0	55.4	20.7	2.2	19.5	2.2
Traveling salesmen, agents, etc	46,142	100.0	82.0	6.3	2.1	5.0	4.6
Craftsmen, foremen, skilled laborers	387,307	100.0	67.8	13.4	1.5	12.4	4.9
Operatives, workers, and laborers	581,007	100.0	53.0	19.9	.9	20.8	5.4
Protective services	39,306	100.0	67.9	10.3	2.7	10.1	9.0
Personal-service workers	157,087	100.0	19.7	37.5	.7	33.4	8.7
Miscellaneous	7,038	100.0	80.2	6.9		11.4	1.5
Occupation not reported	77,097	100.0	18.3	22.1	2.1	25.5	32.0
All occupations	2,157,263	100.0	57.6	17.5	1.4	18.7	4.8
Incorporated places under 5,000:							
Professionals and semiprofessionals	50,900	100.0	61.1	1.2		26.7	11.0
Proprietors, managers, officials [3]	85,266	100.0	71.1	.3	1.4	18.0	9.2
Store and office clerks, salesmen	80,660	100.0	59.7	3.3	1.3	32.0	3.7
Traveling salesmen, agents, etc	6,070	100.0	74.3			18.4	7.3
Craftsmen, foremen, skilled laborers	85,832	100.0	69.4	1.4	1.2	18.2	9.8
Operatives, workers, and laborers	149,181	100.0	61.0	2.3	1.4	28.4	6.9
Protective services	5,460	100.0	64.4			29.5	6.1
Personal-service workers	27,375	100.0	26.6			56.5	16.9
Miscellaneous	843	100.0	87.2				12.8
Occupation not reported	14,165	100.0	24.7			39.1	36.2
All occupations	505,752	100.0	62.6	1.6	1.1	27.4	7.3
Incorporated places of 5,000–24,999:							
Professionals and semiprofessionals	44,053	100.0	67.1	3.5	1.2	25.9	2.3
Proprietors, managers, officials [3]	54,697	100.0	84.0	.8		11.5	3.7
Store and office clerks, salesmen	104,974	100.0	67.3	4.3	1.0	25.5	1.9
Traveling salesmen, agents, etc	8,838	100.0	90.4	1.3		7.0	1.3
Craftsmen, foremen, skilled laborers	88,078	100.0	71.9	3.4	1.4	17.8	5.5
Operatives, workers, and laborers	120,115	100.0	60.2	5.3	.6	27.4	6.5
Protective services	10,042	100.0	80.3	1.1		9.6	9.0
Personal-service workers	27,777	100.0	21.6	11.5	2.6	51.5	12.8
Miscellaneous	1,665	100.0	100.0				
Occupation not reported	18,951	100.0	6.1	3.5		32.2	58.2
All occupations	479,190	100.0	64.4	4.2	.9	24.2	6.3
Incorporated places of 25,000–99,999:							
Professionals and semiprofessionals	39,276	100.0	65.7	13.8	.5	16.8	3.2
Proprietors, managers, officials [3]	46,045	100.0	77.0	7.3	1.0	12.6	2.1
Store and office clerks, salesmen	73,928	100.0	51.3	22.5	1.6	21.3	3.3
Traveling salesmen, agents, etc	17,896	100.0	93.5		2.4	1.2	2.9
Craftsmen, foremen, skilled laborers	82,707	100.0	76.3	9.0	1.4	10.5	2.8
Operatives, workers, and laborers	129,434	100.0	58.9	15.9	.3	20.8	4.1
Protective services	10,272	100.0	78.5	11.3		6.0	4.2
Personal-service workers	25,649	100.0	44.0	23.8	.8	18.1	13.3
Miscellaneous	3,037	100.0	66.1	13.1		20.8	
Occupation not reported	23,392	100.0	16.3	39.6		29.8	14.3
All occupations	451,636	100.0	62.8	15.7	.9	17.2	3.4
Incorporated places of 100,000 and over:							
Professionals and semiprofessionals	65,484	100.0	60.9	25.9	2.1	7.7	3.4
Proprietors, managers, officials [3]	52,619	100.0	76.3	12.5	1.0	7.7	3.1
Store and office clerks, salesmen	164,377	100.0	47.5	38.8	3.6	8.6	1.5
Traveling salesmen, agents, etc	13,338	100.0	64.6	21.1	4.0	2.5	7.8
Craftsmen, foremen, skilled laborers	130,689	100.0	58.7	30.9	1.8	6.2	2.4
Operatives, workers, and laborers	182,278	100.0	37.6	46.5	1.1	10.4	4.4
Protective services	13,532	100.0	51.9	20.4	8.0	5.8	13.9
Personal-service workers	76,286	100.0	8.4	65.1	.1	23.8	2.6
Miscellaneous	1,493	100.0	82.7	5.9		11.4	
Occupation not reported	20,589	100.0	27.3	34.5	7.7	5.2	25.3
All occupations	720,685	100.0	46.4	38.4	2.2	9.9	3.1

[1] Arkansas, Louisiana, North Dakota, Oklahoma, South Dakota, and Wisconsin.
[2] These numbers differ slightly from those shown in tables 8 and 9 because this table includes 36,096 persons for whom occupation was reported, but both the method of transportation and the mileage to work were not reported.
[3] Includes farmers and farm managers requiring home-to-work travel.

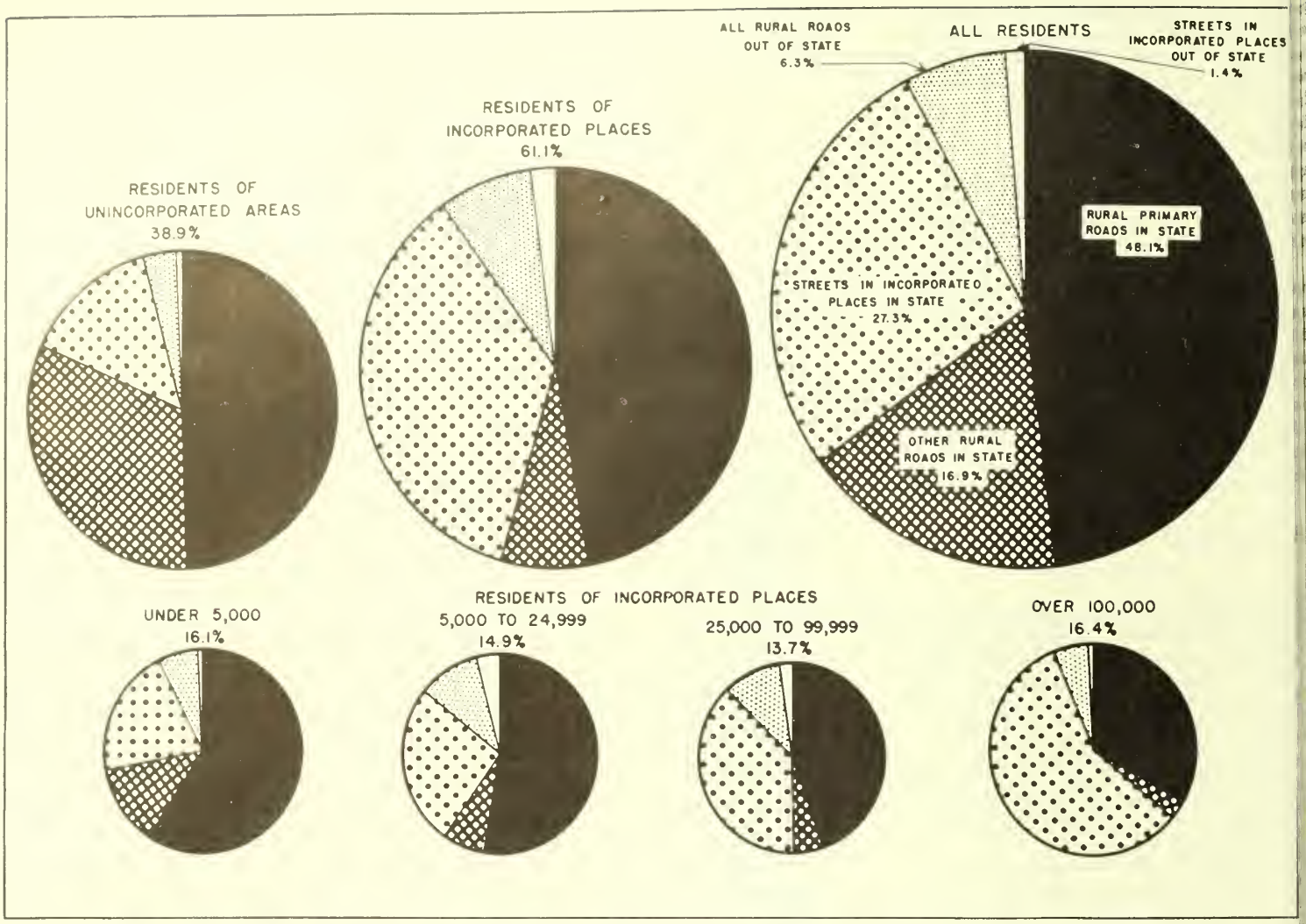
RESIDENTS OF UNINCORPORATED AREAS 38.9%
RESIDENTS OF INCORPORATED PLACES 61.1%
ALL RESIDENTS
ALL RURAL ROADS OUT OF STATE 6.3%
STREETS IN INCORPORATED PLACES OUT OF STATE 1.4%
RURAL PRIMARY ROADS IN STATE 48.1%
STREETS IN INCORPORATED PLACES IN STATE 27.3%
OTHER RURAL ROADS IN STATE 16.9%
RESIDENTS OF INCORPORATED PLACES
UNDER 5,000 16.1%
5,000 TO 24,999 14.9%
25,000 TO 99,999 13.7%
OVER 100,000 16.4%

Figure 11.—Passenger-car travel classified by place of travel and by residence of operator; summer of 1951, four States.

Table 11.—Distribution of passenger-car travel classified by place of travel and by residence of principal operator; summer of 1951, four States [1]

Place of travel	Distribution of passenger-car travel by place of travel, according to residence of principal operator						
	All population groups	Unincorporated areas	Incorporated places with population of—				
			All places	Under 5,000	5,000–24,999	25,000–99,999	100,000 and over
All travel:							
Unincorporated areas:	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Primary highways	54.2	52.9	55.1	65.5	62.8	55.1	37.9
Other rural roads	17.1	32.4	7.3	13.3	7.5	5.0	3.2
Subtotal	71.3	85.3	62.4	78.8	70.3	60.1	41.1
Incorporated places	28.7	14.7	37.6	21.2	29.7	39.9	58.9
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0
In State of residence:							
Unincorporated areas:							
Primary highways	48.1	49.4	47.3	59.2	52.8	45.0	32.6
Other rural roads	16.9	32.3	7.1	13.0	6.9	4.9	3.1
Subtotal	65.0	81.7	54.4	72.2	59.7	49.9	35.7
Incorporated places	27.3	14.2	35.7	20.5	26.0	37.3	58.1
Total	92.3	95.9	90.1	92.7	85.7	87.2	93.8
Outside State of residence:							
Unincorporated areas:							
Primary highways	6.1	3.5	7.8	6.3	10.0	10.1	5.3
Other rural roads	.2	.1	.2	.3	.6	.1	.1
Subtotal	6.3	3.6	8.0	6.6	10.6	10.2	5.4
Incorporated places	1.4	.5	1.9	.7	3.7	2.6	.8
Total	7.7	4.1	9.9	7.3	14.3	12.8	6.2

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.

incorporated places were very similar. Th distribution for unincorporated areas varie more widely because of the large number c farmers and farm managers, who owned 4 percent of all cars but drove only 39 percen of the total travel.

From figure 14 it may be deduced tha where the bars indicating percentage of trave and percentage of vehicles in use are of equa length, the vehicle operators drove about th average miles for the entire sample; where th travel bar is the shorter of two, the amoun of travel was less than the average; and wher the travel bar is the longer, the amount o travel was above the average. Since the dat in this report are based primarily on studie covering only one season of the year, figure on the average annual miles traveled pe vehicle are omitted.

AUTOMOBILE TRIP-LENGTH CHARACTERISTICS

Automobile trips were found to be pre dominantly short-distance travel, that is trips having one-way trip length of less tha five miles. The trip-length analyses of th

iginal road-use studies,[4] which excluded ıose trips entirely within incorporated places, ıowed that about 38 percent of the trips ere under 5 miles in length. As indicated figure 15 and table 14, the present studies, hich include trips entirely within the incor-ɔrated places, show that 62 percent of all ıssenger-car trips were less than 5 miles in ngth; 81 percent were less than 10 miles in ngth. The average length of all trips in ıese four States was 8.3 miles.

While distributions of travel by trip length ere not developed for all the States in the ırly analyses of the motor-vehicle-use data, ı two States—Louisiana and Oklahoma—pproximately 13 percent of the passenger-car ·avel was found to result from the large umber of trips of less than 5 miles in length. lmost one-half of the travel resulted from ·ips 30 miles or greater in length. This in-ɔrmation is shown in table 15.

Distribution by Place of Residence

It may be seen in figure 15 that the short ɔassenger-car trips predominate in all popula-ion groups. The residents of unincorporated ·eas performed a slightly lesser proportion f their trips in the under-5-mile-length cate-ɔory, but they still accounted for 48 percent f the total. The highest percentage of trips n the second category, 5–9 miles in length, vas performed by the residents of the unin-ɔrporated areas. All of the places of resi-lence showed nearly the same proportion—rom 76 to 88 percent—for all passenger-car ·ips of less than 10 miles in length. Each of he four States reporting data showed a high legree of similarity in the trip-length fre-uency distribution.

Generally, the average length of automobile ·ips made by residents of the rural areas was slightly greater than for residents of incor-ɔorated places: 9.0 miles for unincorporated ınd 7.9 miles for incorporated places.

Among the four States, the average length ɔf passenger-car trip for all population groups ɔombined ranged from 7.9 miles in Wisconsin ·o 9.3 miles in Louisiana.

Distribution by Occupational Group

Occupational-group characteristics were ob-tained for passenger-car trips, and are shown in figure 16 and table 16. A predominant occurrence of the short trip is noted in each occupational group; this is similar to the frequency of short trips observed for residents of the various population groups. Farmers and farm managers and traveling salesmen and agents had the lowest proportion of trips under 5 miles—although they still made 50 to 60 percent of their trips in this length category. Farmers had somewhat more trips than other occupations in the 5–9-mile-length group so that each of the occupational groups had about the same percentage of trips of less than 10 miles. The traveling salesmen and agents reported a slightly greater proportion of the longer trips than did other occupations. It should be noted that the occupational breakdown includes all trips made in vehicles classified according to the occupation of the principal operator and, of course, includes the trips made in that vehicle when other members of the family were driving. For example, if a farmer's wife uses a car of which he is the principal operator, such trips are included in those of the farmer and farm manager group.

The average trip length in the various occupational classifications varied from 7.6 miles for store and office clerks and salesmen, personal-service workers, and protective-service workers to 12.2 miles for traveling salesmen and agents. As already indicated, the average trip length for all occupations combined was 8.3 miles.

The occupational group which reported the largest number of trips, 21 percent of the total, was that including operatives and semi-skilled and unskilled laborers. It is not surprising to note also that this group owned 20 percent of all cars. The next largest number of trips was reported by craftsmen, foremen, and skilled laborers, where 18 percent were so reported; this occupational group owned 17 percent of all the cars.

Farmers and farm managers reported 16 percent of all the trips and owned 20 percent of all the cars. Here the range among the States was much wider—from 8 percent in Louisiana to 22 percent in Wisconsin. These percentages are in the same ratio as the range in the reported car ownership among the four States.

Generally, it may be said that the percentage of trips in each occupational group was closely related to the percentage of automobiles in these same groups. The higher-salaried workers and traveling salesmen and agents reported more than the average number of trips per car and the farmers and farm managers less than the average number.

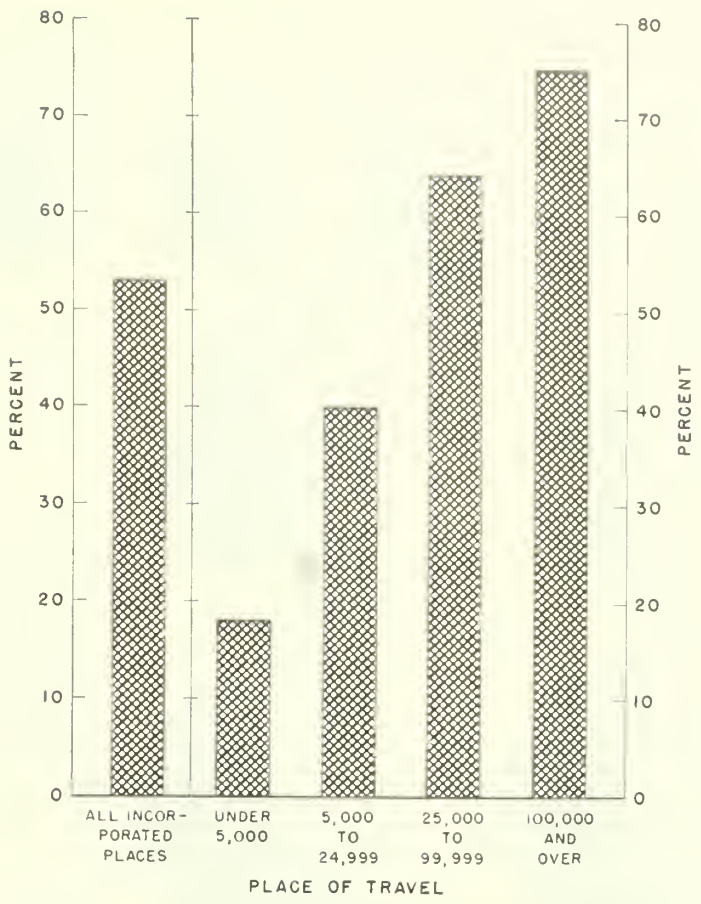

Figure 12.—Proportion of total passenger-car travel in incorporated places by residents of the State resulting from trips confined entirely within these places; summer of 1951, four States.

Table 12.—Distribution of passenger-car travel in incorporated places by residents of the State, resulting from trips confined entirely within these places, classified by place of travel and State; summer of 1951, four States [1]

Place of travel	Four States	Arkansas	Louisiana	Oklahoma	Wisconsin
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Under 5,000	18	16	23	17	17
5,000–24,999	40	43	46	41	36
25,000–99,999	64	77	65	75	60
100,000 and over	75	82	69	80	76
All places	53	55	57	60	48

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.

[4] *Preliminary results of road-use studies*, by R. H. Paddock and R. P. Rodgers. PUBLIC ROADS, vol. 20, No. 3, May 1939, p. 49.

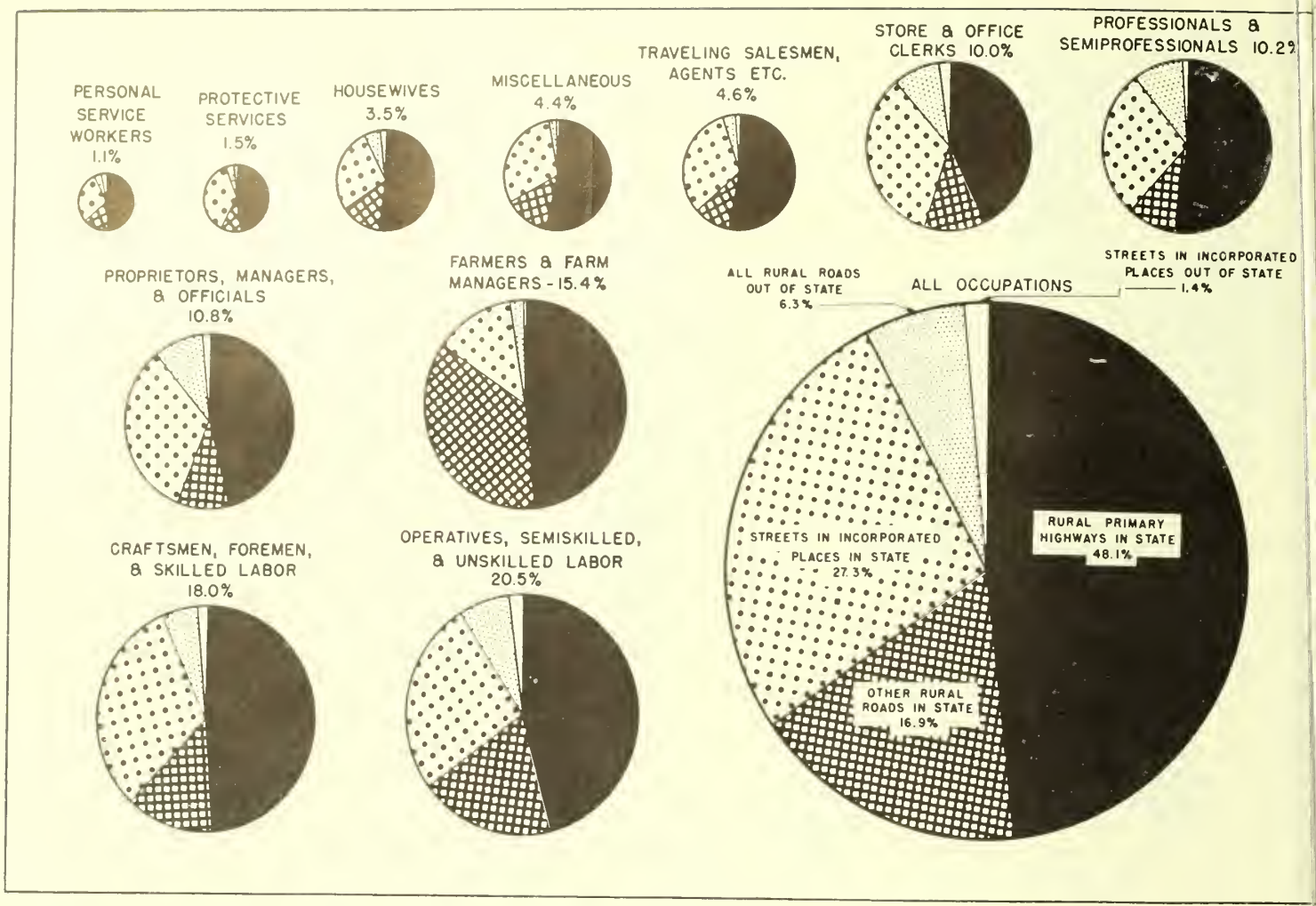
PERSONAL SERVICE WORKERS 1.1%
PROTECTIVE SERVICES 1.5%
HOUSEWIVES 3.5%
MISCELLANEOUS 4.4%
TRAVELING SALESMEN, AGENTS ETC. 4.6%
STORE & OFFICE CLERKS 10.0%
PROFESSIONALS & SEMIPROFESSIONALS 10.2%
PROPRIETORS, MANAGERS, & OFFICIALS 10.8%
FARMERS & FARM MANAGERS - 15.4%
ALL RURAL ROADS OUT OF STATE 6.3%
ALL OCCUPATIONS
STREETS IN INCORPORATED PLACES OUT OF STATE 1.4%
CRAFTSMEN, FOREMEN, & SKILLED LABOR 18.0%
OPERATIVES, SEMISKILLED, & UNSKILLED LABOR 20.5%
RURAL PRIMARY HIGHWAYS IN STATE 48.1%
STREETS IN INCORPORATED PLACES IN STATE 27.3%
OTHER RURAL ROADS IN STATE 16.9%

Figure 13.—Passenger-car travel classified by place of travel and by occupation of principal operator; summer of 1951, four States.

Table 13.—Distribution of passenger-car travel classified by place of travel and by occupation of principal operator; summer of 19 four States [1]

Place of travel	Distribution of passenger-car travel by place of travel, according to occupation of principal operator											
	All occupations	Professionals and semiprofessionals	Proprietors, managers, and officials	Farmers and farm managers	Store and office clerks, salesmen	Traveling salesmen, agents, etc.	Craftsmen, foremen, skilled laborers, etc.	Operatives, semiskilled and unskilled workers, and laborers	Protective services	Personal-service workers	Housewives	Miscellaneous
All travel:	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
In unincorporated areas:												
Primary highways	54.2	61.0	55.9	49.9	52.0	56.8	54.5	53.1	51.6	53.1	56.8	55.1
Other rural roads	17.1	10.1	9.8	36.8	12.0	10.3	13.0	18.9	9.9	15.3	14.0	14.2
Subtotal	71.3	71.1	65.7	86.7	64.0	67.1	67.5	72.0	61.5	68.4	70.8	69.3
In incorporated places	28.7	28.9	34.3	13.3	36.0	32.9	32.5	28.0	38.5	31.6	29.2	30.7
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
In State of residence:												
In unincorporated areas:												
Primary highways	48.1	52.2	46.2	48.2	42.8	53.0	49.0	46.0	47.9	48.8	52.0	53.3
Other rural roads	16.9	9.3	9.7	36.5	11.9	10.3	12.9	18.8	9.9	15.0	13.6	13.8
Subtotal	65.0	61.5	55.9	84.7	54.7	63.3	61.9	64.8	57.8	63.8	65.6	67.1
In incorporated places	27.3	27.6	32.9	12.8	33.8	31.6	31.4	26.2	36.5	28.8	26.9	30.2
Total	92.3	89.1	88.8	97.5	88.5	94.9	93.3	91.0	94.3	92.6	92.5	97.3
Outside State of residence:												
In unincorporated areas:												
Primary highways	6.1	8.8	9.7	1.7	9.2	3.8	5.5	7.1	3.7	4.3	4.8	1.8
Other rural roads	.2	.8	.1	.3	.1	------	.1	.1	------	.3	.4	.4
Subtotal	6.3	9.6	9.8	2.0	9.3	3.8	5.6	7.2	3.7	4.6	5.2	2.2
In incorporated places	1.4	1.3	1.4	.5	2.2	1.3	1.1	1.8	2.0	2.8	2.3	.5
Total	7.7	10.9	11.2	2.5	11.5	5.1	6.7	9.0	5.7	7.4	7.5	2.7

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.
[2] Miscellaneous includes retired persons, occupations not otherwise classified, and occupations not reported.

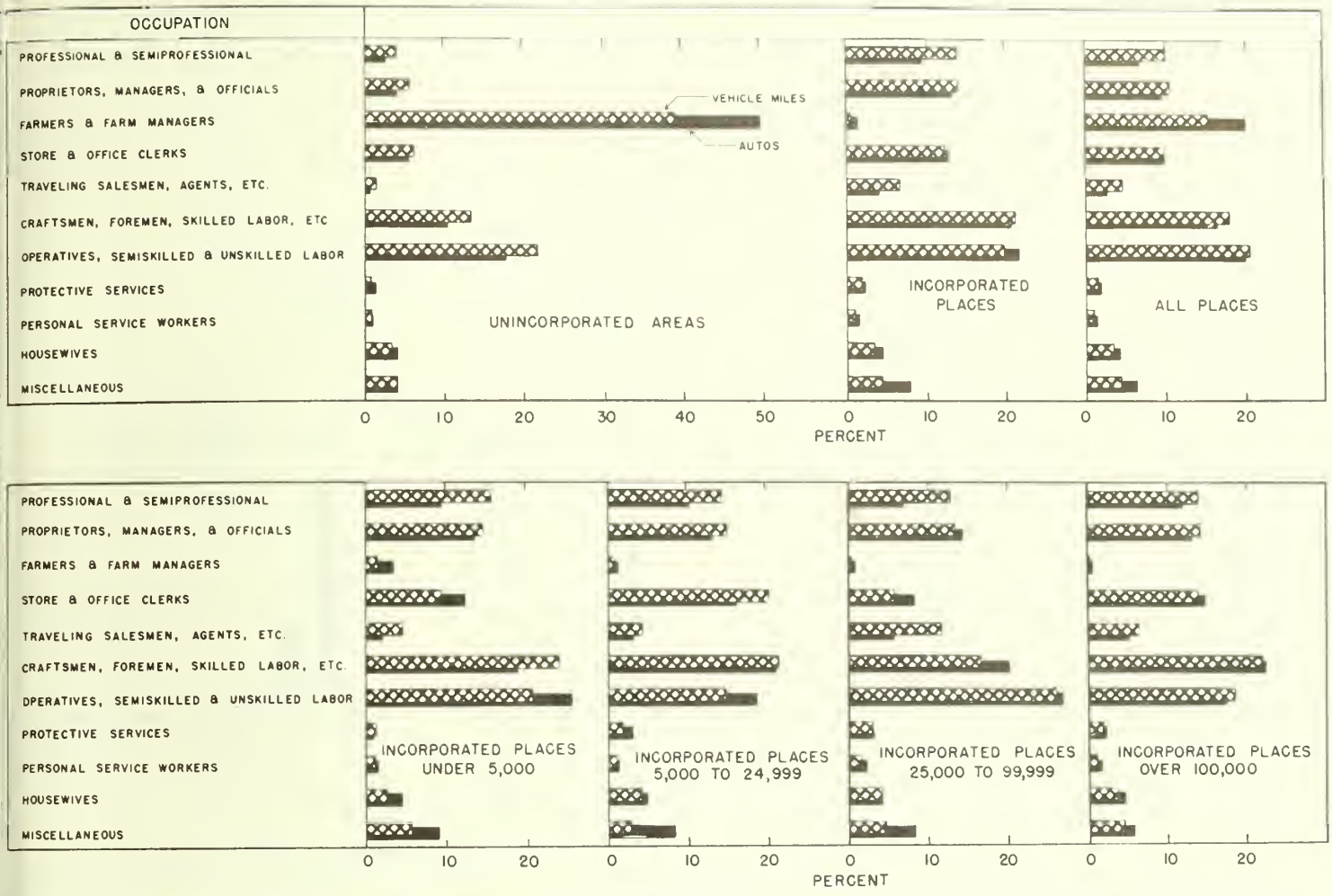

Figure 14.—Passenger cars in use and passenger-car travel, classified by occupation for each place of residence of principal operator; summer of 1951, four States.

Distribution by Purpose of Trip

The following discussion relates trip-length frequency to purpose of travel. A more complete discussion of purpose of travel as related to amount of travel by occupation and residence of the principal operator is given in the section of the report following this discussion.

The trip-length frequency distribution for purposes other than vacation trips and to a lesser extent trips for medical and dental purposes was very similar, with a high frequency of short-length trips. This information may be seen in figure 17 and table 17.

Vacation trips are, of course, predominantly in the longest-length group of 100 miles or more. This length group includes about 69 percent of the vacation trips. In respect to vacation trips, it is somewhat odd to note that one-tenth of the trips for this purpose were found to be under 5 miles in length. This is probably caused largely by the number of people residing close to the place where they spend their vacation.

Trips for medical and dental purposes, while following generally the trip-length distribution of purposes other than vacation trips, have proportionately greater percentages of trips in the several length groups above 5 miles, although 42 percent of these trips were in the under-5-mile group. This, of course, is a result of the increasing centralization of medical services in the larger places.

Shopping trips exhibited the greatest proportion of short-length trips (under 5 miles), 76 percent, as compared with 74 percent for educational, civic, and religious trips, 68 percent for work trips, and about 50 percent for each of the remaining purposes other than vacation trips and medical and dental trips.

PURPOSE OF AUTOMOBILE TRAVEL

The purpose for which the travel was performed is considered an important characteristic of any travel-habit study. In this report, purpose of travel is limited to passenger-car travel since most of the truck travel is performed for business purposes. All travel in passenger cars was classified according to the purpose for which the vehicle trip was made. For example, if the vehicle was being used to carry a passenger to work, the trip was classified as a work trip rather than a trip to serve a passenger, as would be done in most urban origin-and-destination studies. In this study, serving a passenger is considered only an incidental purpose and not the reason for which the vehicle is used. Also, returning home is not considered as a separate trip purpose in these studies. The return-home portion of the trip is classified according to the purpose of the outbound portion of the trip or, in the case of a round-trip involving three or more principal segments, the return-home portion is classified according to the major purpose involved for all segments.

Distribution of Trips and Travel by Purpose

The distribution of the number of trips and the amount of travel according to trip purpose is shown in figure 18 and table 18. Roughly two-thirds of the total travel—73 percent of the trips and 60 percent of the vehicle-miles—was "necessity" travel, that is, travel made in connection with earning a living and for family business; and of the necessity travel itself about three-fourths of the vehicle-miles and two-thirds of the trips were made in connection with earning a living. One-half of the vehicle-miles of travel and two-thirds of the trips made in connection with earning a living resulted from travel to and from work.

About 5 percent of the trips and 3 percent of the travel was for educational, civic, and religious purposes. The remainder, about one-fourth of the trips and slightly over one-third of the travel, was made for social and recreational purposes.

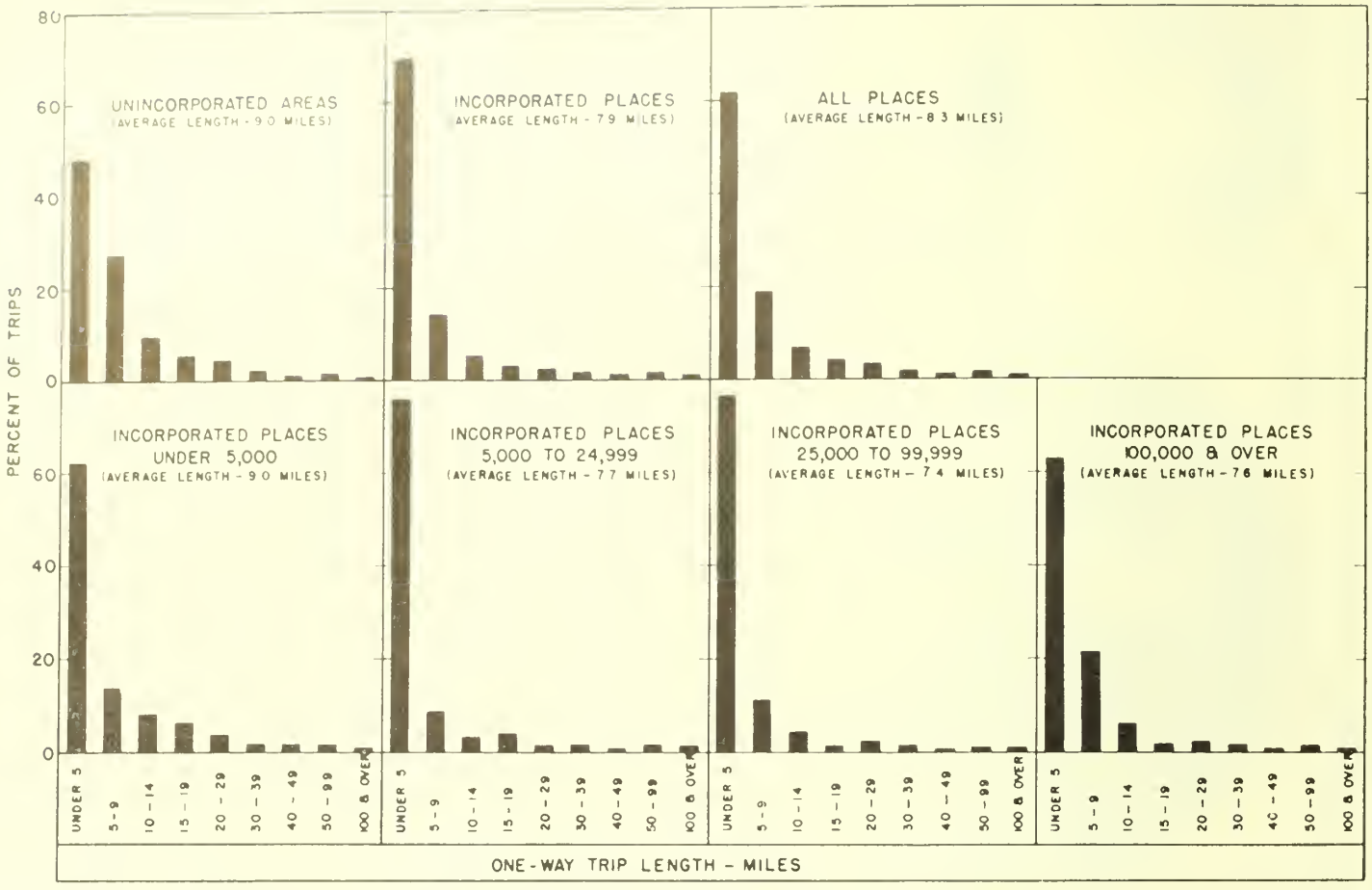

Figure 15.—Passenger-car trips classified by trip length and by residence of principal operator; summer of 1951, four States.

The average trip length for each purpose is shown at the top of the bars in figure 18. As noted previously, the shopping trip was the shortest trip and the vacation trip was by far the longest. The distributions of travel in each of the four States were very similar for all the principal purposes. The only differences of any magnitude occurred in the distribution between pleasure rides and "other" under the social and recreational purpose classification. Two of the States showed a low percentage of pleasure rides and a high percentage of "other" while the other two States showed the reverse. This was the result of differences in the interpretation placed by the States on the definition of a "pleasure ride".

Relation of Travel Purpose to Residence and Occupation of Operator

As shown in figure 19 and table 19, the distribution of passenger-car travel by trip purpose was generally similar for each population group of residence. For example, each group, with one exception, performed about 45 percent of their total automobile travel in connection with earning a living. The only deviation occurred in the places having a population of 25,000–99,999 in which 34 percent of the travel was for earning a living. This population group had a correspondingly high percentage of travel (50 percent) for social and recreational purposes.

Family business accounted for a greater proportion of the total travel by residents of

Table 14.—Distribution of passenger-car trips classified by one-way trip length and by residence of principal operator; summer of 1951, four States [1]

One-way trip length	Distribution of passenger-car trips by trip length, according to residence of principal operator						
	All places	Unincorporated areas	Incorporated places with population of—				
			Total	Under 5,000	5,000–24,999	25,000–99,999	100,000 and over
Miles	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Under 5	61.7	48.0	69.3	62.3	75.9	76.4	63.0
5–9	18.9	27.3	14.2	13.9	8.8	11.4	21.6
10–14	6.9	9.6	5.5	8.0	3.3	4.4	6.1
15–19	4.2	5.6	3.2	6.2	4.1	1.3	2.1
20–29	3.2	4.5	2.5	3.5	1.8	2.2	2.5
30–39	1.8	2.2	1.7	1.6	1.9	1.5	1.6
40–49	1.0	1.0	1.0	1.7	1.0	.6	.7
50–99	1.5	1.3	1.6	1.7	1.9	1.1	1.6
100 and over	.8	.5	1.0	1.1	1.3	1.1	.8
All trips	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Average length, miles	*8.3*	*9.0*	*7.9*	*9.0*	*7.7*	*7.4*	*7.6*

Arkansas, Louisiana, Oklahoma, and Wisconsin.

Table 15.—Distribution of passenger-car travel classified by one-way trip length and by residence of principal operator; summer of 1951, two States [1]

One-way trip length	Distribution of passenger-car travel by trip length, according to residence of principal operator						
	All places	Unincorporated areas	Incorporated places with population of—				
			Total	Under 5,000	5,000–24,999	25,000–99,999	100,000 and over
Miles	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Under 5	13.4	9.6	15.8	8.3	14.2	20.8	21.7
5–9	14.2	15.7	13.3	9.7	8.2	10.0	20.5
10–14	7.9	9.4	7.0	6.7	5.4	5.5	8.8
15–19	9.5	11.1	8.5	11.3	15.5	2.7	3.0
20–29	10.0	13.9	7.5	9.1	6.6	7.5	6.9
30–39	7.6	9.8	6.1	7.2	5.6	10.9	4.3
40–49	4.3	5.1	3.7	3.1	6.1	5.6	1.9
50–99	13.9	11.3	15.6	14.9	15.5	18.5	15.3
100 and over	19.2	14.1	22.5	29.7	22.9	18.5	17.6
All travel	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Louisiana and Oklahoma.

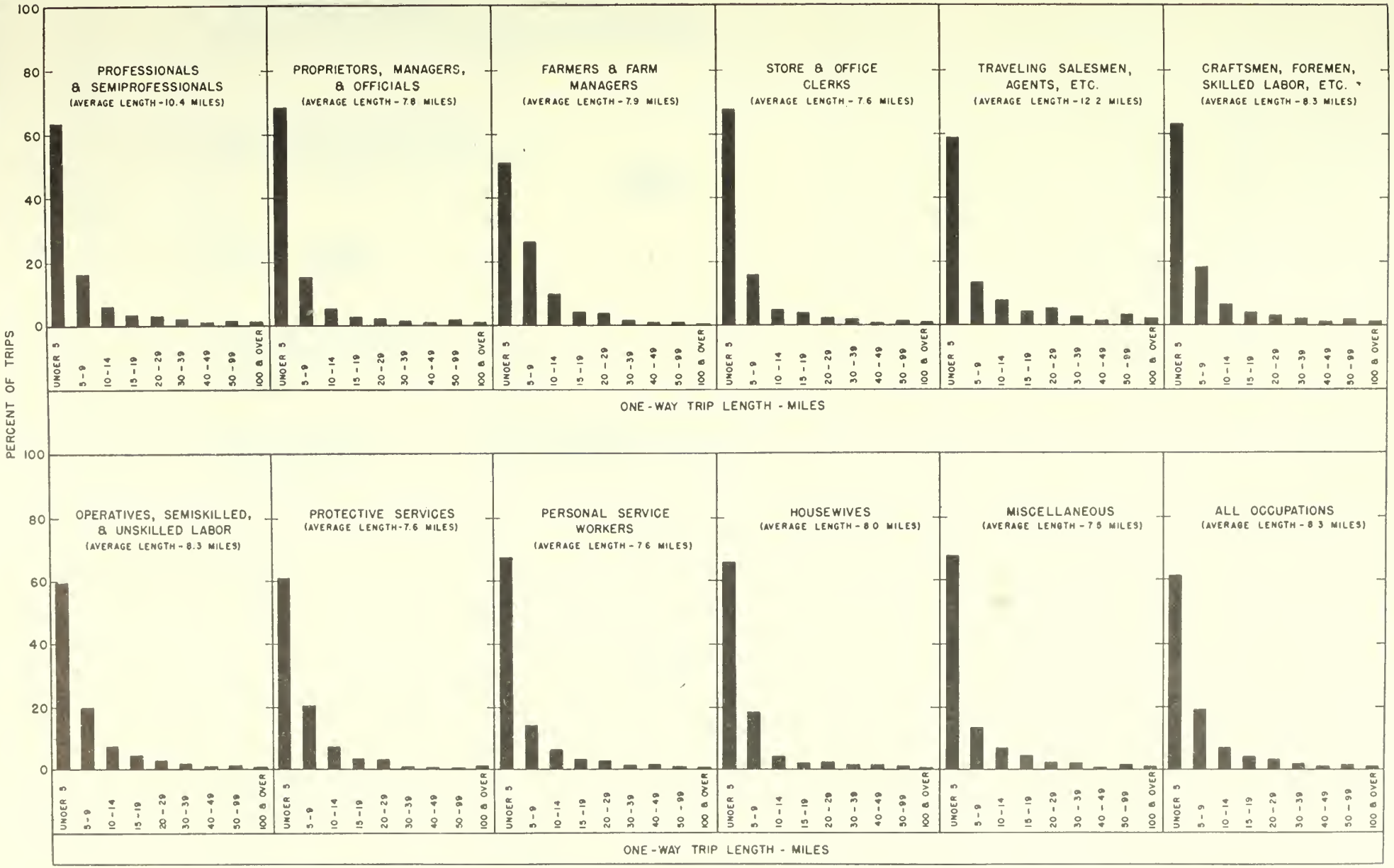
PERCENT OF TRIPS
PROFESSIONALS & SEMIPROFESSIONALS (AVERAGE LENGTH - 10.4 MILES)
PROPRIETORS, MANAGERS, & OFFICIALS (AVERAGE LENGTH - 7.8 MILES)
FARMERS & FARM MANAGERS (AVERAGE LENGTH - 7.9 MILES)
STORE & OFFICE CLERKS (AVERAGE LENGTH - 7.6 MILES)
TRAVELING SALESMEN, AGENTS, ETC. (AVERAGE LENGTH - 12.2 MILES)
CRAFTSMEN, FOREMEN, SKILLED LABOR, ETC. (AVERAGE LENGTH - 8.3 MILES)
OPERATIVES, SEMISKILLED, & UNSKILLED LABOR (AVERAGE LENGTH - 8.3 MILES)
PROTECTIVE SERVICES (AVERAGE LENGTH - 7.6 MILES)
PERSONAL SERVICE WORKERS (AVERAGE LENGTH - 7.6 MILES)
HOUSEWIVES (AVERAGE LENGTH - 8.0 MILES)
MISCELLANEOUS (AVERAGE LENGTH - 7.5 MILES)
ALL OCCUPATIONS (AVERAGE LENGTH - 8.3 MILES)
0
20
40
60
80
100
UNDER 5
5 - 9
10 - 14
15 - 19
20 - 29
30 - 39
40 - 49
50 - 99
100 & OVER
ONE - WAY TRIP LENGTH - MILES

Figure 16.—Passenger-car trips classified by trip length and by occupation of principal operator; summer of 1951, four States.

Table 16.—Distribution of passenger-car trips classified by one-way trip length and by occupation of principal operator; summer of 19[illegible] four States [1]

Characteristic	All occupations	Professionals and semiprofessionals	Proprietors, managers, and officials	Farmers and farm managers	Store and office clerks, salesmen	Traveling salesmen, agents, etc.	Craftsmen, foremen, skilled laborers, etc.	Operatives, semiskilled and unskilled workers, and laborers	Protective services	Personal service workers	Housewives	Miscellaneous [2]
	Distribution of passenger-car trips by trip length and occupation of principal operator											
	Distribution by One-Way Trip Length											
Miles	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Under 5	61.7	63.3	68.5	51.0	67.5	58.8	62.8	59.5	61.0	67.6	66.0	67.9
5–9	18.9	16.4	15.3	26.6	16.0	13.6	18.2	20.1	20.4	14.2	18.6	13.5
10–14	6.9	6.3	5.3	10.0	5.0	7.9	6.5	7.4	7.4	6.8	4.4	7.2
15–19	4.2	3.8	2.9	4.5	4.0	4.5	4.1	5.1	4.0	3.4	2.4	4.6
20–29	3.2	3.2	2.6	4.0	2.6	5.3	3.2	3.2	3.4	3.1	2.7	2.3
30–39	1.8	2.2	1.3	1.9	1.8	3.0	1.9	1.8	.8	1.4	1.7	2.1
40–49	1.0	1.3	1.0	.8	.9	1.4	.8	1.0	.8	1.8	1.9	.3
50–99	1.5	1.9	2.0	1.0	1.3	3.3	1.6	1.3	.6	.9	1.3	1.2
100 and over	.8	1.6	1.1	.2	.9	2.2	.9	.6	1.6	.8	1.0	.9
All trips	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Average length, miles	*8.3*	*10.4*	*7.8*	*7.9*	*7.6*	*12.2*	*8.3*	*8.3*	*7.6*	*7.6*	*8.0*	*7.5*
	Distribution of Trips by Occupation											
Arkansas	100.0	11.2	15.4	11.2	11.5	1.9	17.2	19.0	1.9	1.0	4.0	5.7
Louisiana	100.0	10.4	12.8	8.2	12.5	2.7	20.0	19.8	2.1	1.7	4.8	5.0
Oklahoma	100.0	7.7	9.7	12.1	17.3	4.5	15.9	19.3	2.7	1.1	3.5	6.2
Wisconsin	100.0	6.7	11.0	22.4	7.3	3.0	18.6	21.8	.9	1.1	3.3	3.9
Four States combined	100.0	8.1	11.6	16.3	10.9	3.1	18.1	20.6	1.6	1.2	3.7	4.8

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.
[2] Includes retired persons, occupations not otherwise classified, and occupations not reported.

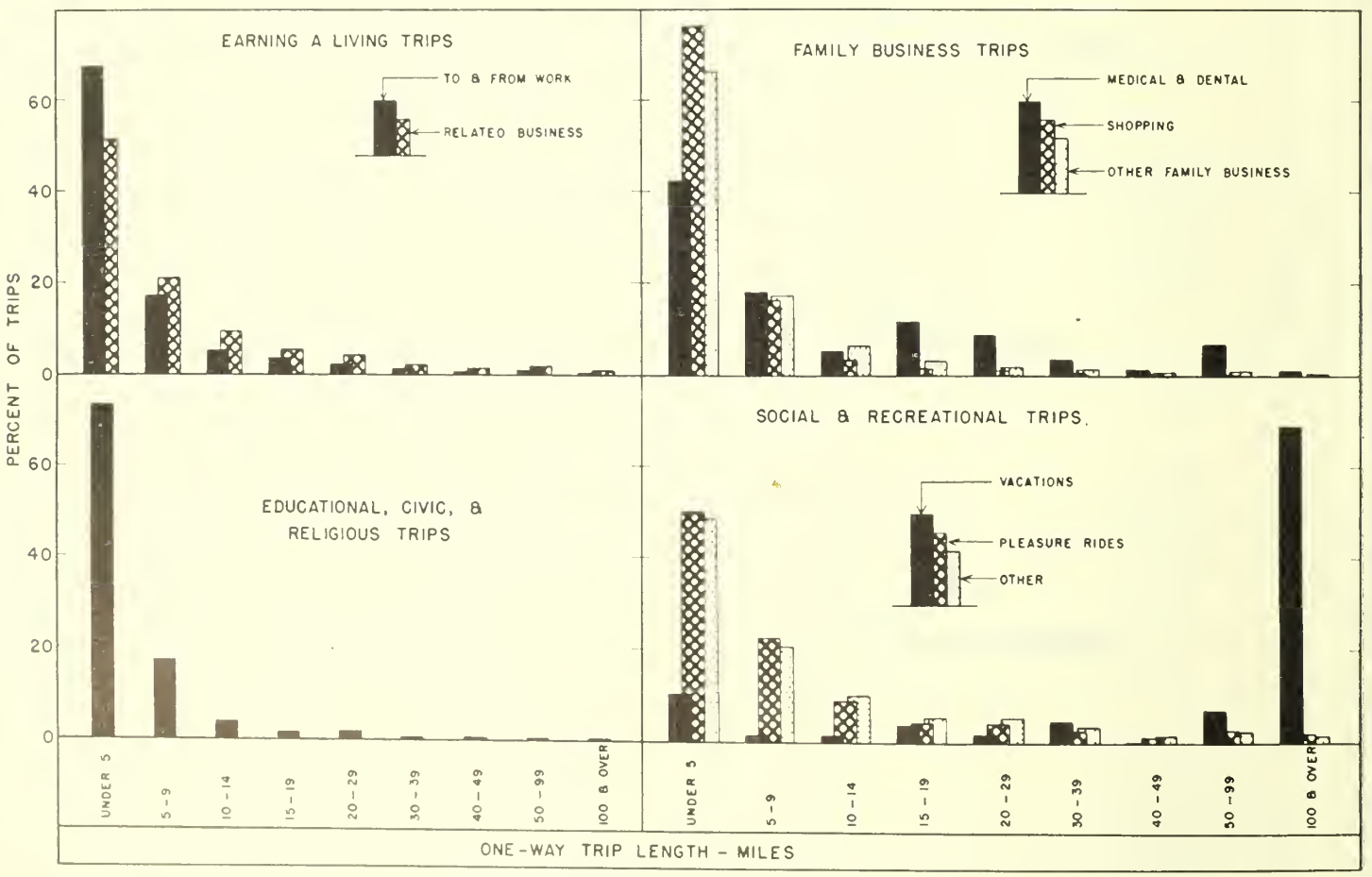

Figure 17.—Passenger-car trips classified by trip length and by major purpose of trip; summer of 1951, four States.

Table 17.—Distribution of passenger-car trips classified by trip length and by purpose of trip; summer of 1951, four States [1]

One-way trip length	Distribution of passenger-car trips by trip length and purpose									
	All purposes	Earning a living		Family business			Educational, civic, religious	Social and recreational		
		To and from work	Related business	Medical and dental	Shopping	Other		Vacations	Pleasure rides	Other
Purpose by Trip Length										
Miles	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Under 5	61.7	67.6	51.6	42.3	76.1	66.5	73.5	10.7	50.3	48.7
5–9	18.9	17.4	21.2	18.1	16.7	17.4	17.4	1.3	22.8	20.8
10–14	6.9	5.5	9.7	5.3	3.3	6.7	4.0	1.5	9.1	10.5
15–19	4.2	3.8	5.6	12.0	1.9	3.3	1.8	3.8	4.6	5.7
20–29	3.2	2.4	4.6	9.0	1.1	2.1	2.0	1.9	4.4	5.3
30–39	1.8	1.3	2.5	3.9	.6	1.7	.3	4.8	2.7	3.4
40–49	1.0	.8	1.6	1.3	.2	.8	.3		1.2	1.6
50–99	1.5	.9	2.1	7.0	.1	1.0	.4	7.1	2.8	2.7
100 and over	.8	.3	1.1	1.1		.5	.3	68.9	2.1	1.3
All trips	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Trip Length by Purpose										
Under 5	100.0	35.4	14.2	.9	16.5	9.4	5.5		7.4	10.7
5–9	100.0	29.7	19.1	1.3	11.8	8.1	4.3		10.9	14.8
10–14	100.0	25.5	23.7	1.0	6.4	8.5	2.7		11.8	20.4
15–19	100.0	30.0	22.7	3.9	6.3	7.0	2.0	.2	10.0	17.9
20–29	100.0	24.1	24.1	3.7	4.5	5.6	2.9	.1	12.4	22.6
30–39	100.0	23.2	22.6	2.8	4.1	7.9	.7	.5	13.4	24.8
40–49	100.0	26.8	28.5	1.8	2.1	7.0	1.2		10.8	21.8
50–99	100.0	18.7	24.2	6.3	1.3	5.7	1.4	.9	17.0	24.5
100 and over	100.0	9.3	23.0	1.8	.7	5.6	2.0	14.5	23.1	20.0
All trips	100.0	32.2	17.0	1.3	13.4	8.7	4.6	.2	9.0	13.6

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.

Table 18.—Distribution of passenger-car trips and travel classified by purpose of trip; summer of 1951, four States [1]

Purpose of trip	Distribution of—		Average trip length
	Trips	Vehicle miles	
	Pct.	*Pct.*	*Miles*
Earning a living:			
To and from work	32.2	22.7	5.9
Related business	17.0	21.3	10.4
Subtotal	49.2	44.0	7.4
Family business:			
Medical and dental	1.3	2.5	15.5
Shopping	13.4	6.0	3.7
Other	8.7	7.3	6.9
Subtotal	23.4	15.8	5.6
Education, civic, and religious	4.6	2.8	5.0
Social and recreational:			
Vacations	.2	4.8	249.7
Pleasure rides	9.0	14.1	13.0
Other	13.6	18.5	11.4
Subtotal	22.8	37.4	13.7
All purposes	100.0	100.0	8.3

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.

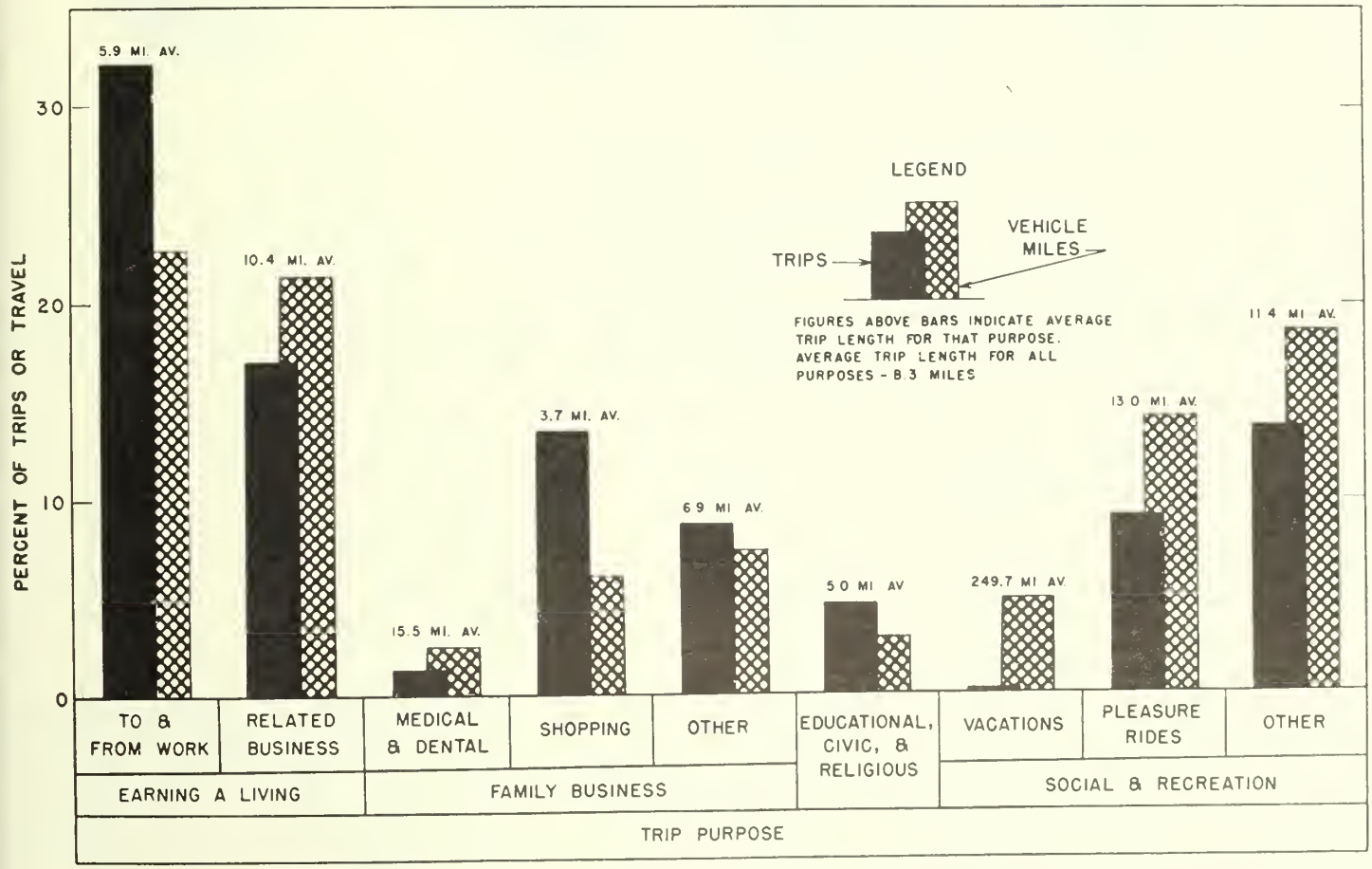

Figure 18.—Passenger-car trips and travel classified by major purpose of trip; summer of 1951, four States.

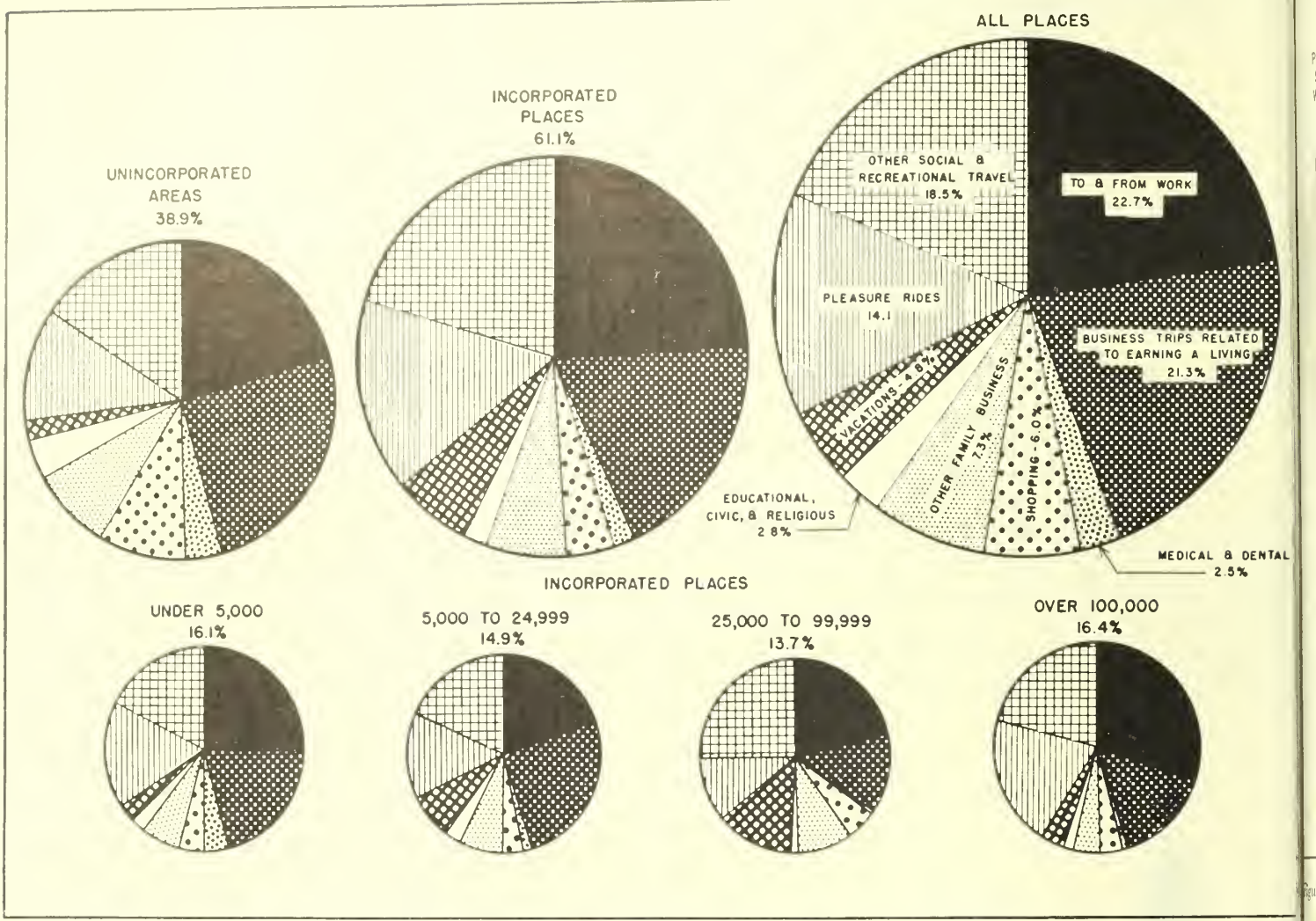

Figure 19.—Passenger-car travel classified by major purpose of travel and by residence of principal operator; summer of 1951, four State

unincorporated areas (21 percent) than it did for residents of incorporated places (12 percent). On the other hand, a higher proportion of travel for social and recreational purposes was found for the residents of incorporated places (43 percent) than of unincorporated areas (29 percent).

The distribution of passenger-car travel by purpose of travel, classified by the occupation of the principal operator, as shown in figure 20 and table 20, was fairly similar for nearly all occupations.

Necessity travel ranged from 47 to 72 percent for gainfully employed workers in the occupational groups, averaging 60 percent.

Farmers did little home-to-work travel but had more than average related business travel, so they had nearly the same amount of total travel for earning a living as was found for other occupations.

Cars of which traveling salesmen and agents were the principal operators traveled 67 percent of their total mileage for the purpose of earning a living, whereas cars operated by persons in other occupations, excluding housewives and personal-service workers, traveled from 41 to 59 percent in this category.

Cars of which the housewives were the principal operators had only one-fifth of their mileage as earning-a-living travel; nearly one-half of their travel was for social a recreational purposes.

Table 19.—Distribution of passenger-car travel classified by purpose of travel and residence of principal operator; summer of 1951, four States [1]

Purpose of travel	Distribution of passenger-car travel by purpose, according to residence of principal operator						
	All population groups	Unincorporated areas	Incorporated places with population of—				
			All places	Under 5,000	5,000–24,999	25,000–99,999	100,000 and over
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Earning a living:							
To and from work	22.7	20.6	24.1	25.0	19.6	21.0	29.8
Related business	21.3	24.9	19.0	21.2	25.7	13.3	15.4
Subtotal	44.0	45.5	43.1	46.2	45.3	34.3	45.2
Family business:							
Medical and dental	2.5	3.8	1.7	3.9	1.4	.5	.8
Shopping	6.0	9.1	4.0	4.0	3.3	5.2	3.6
Other	7.3	8.4	6.5	6.3	7.2	9.2	4.0
Subtotal	15.8	21.3	12.2	14.2	11.9	14.9	8.4
Educational, civic, religious	2.8	4.0	2.0	2.2	2.6	1.2	1.8
Social and recreational:							
Vacation	4.8	2.1	6.5	2.8	7.4	13.3	3.6
Pleasure rides	14.1	11.5	15.8	17.0	14.2	10.9	20.1
Other	18.5	15.6	20.4	17.6	18.6	25.4	20.9
Subtotal	37.4	29.2	42.7	37.4	40.2	49.6	44.6
All purposes	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.

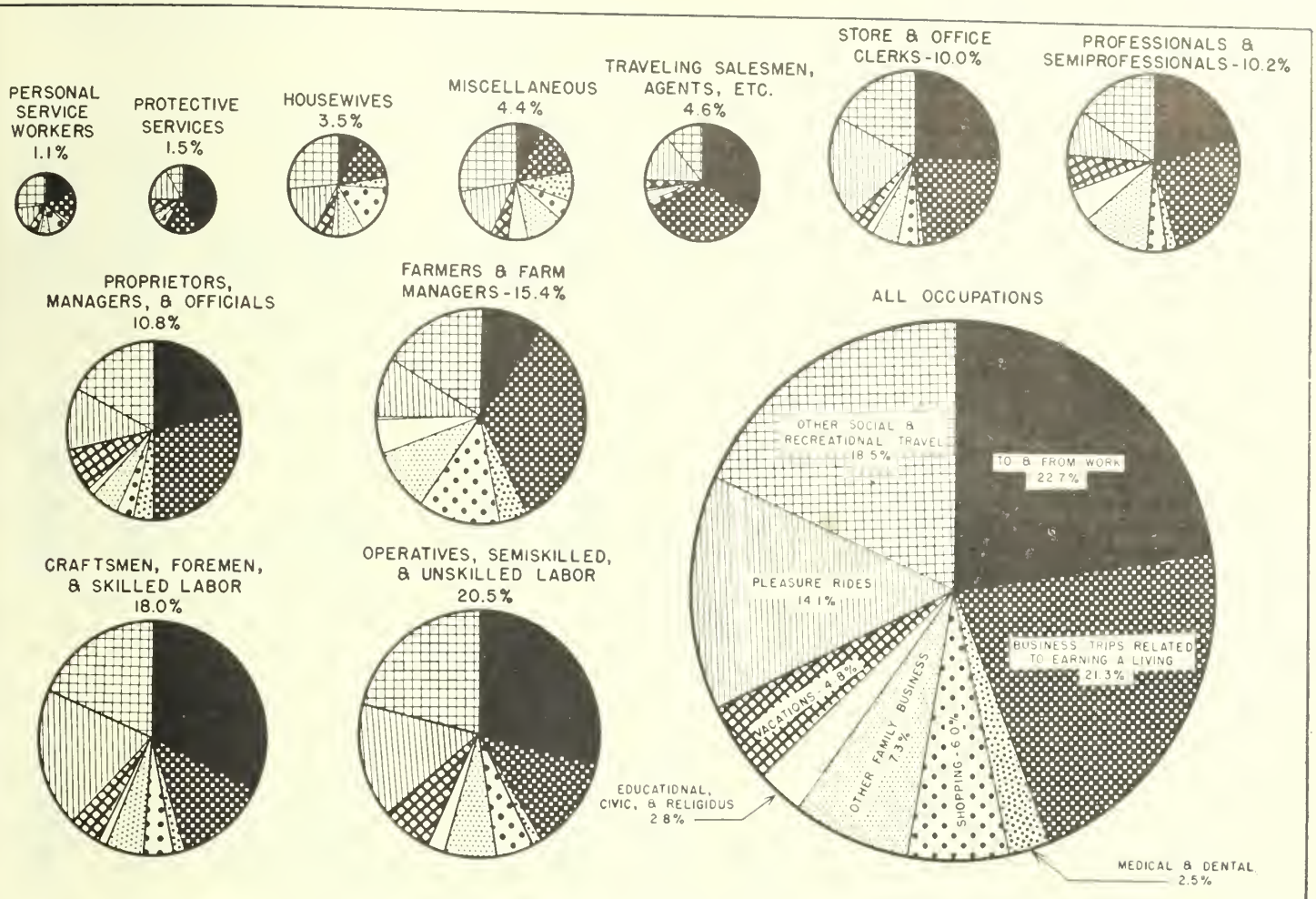
PERSONAL SERVICE WORKERS 1.1%
PROTECTIVE SERVICES 1.5%
HOUSEWIVES 3.5%
MISCELLANEOUS 4.4%
TRAVELING SALESMEN, AGENTS, ETC. 4.6%
STORE & OFFICE CLERKS - 10.0%
PROFESSIONALS & SEMIPROFESSIONALS - 10.2%
PROPRIETORS, MANAGERS, & OFFICIALS 10.8%
FARMERS & FARM MANAGERS - 15.4%
ALL OCCUPATIONS
CRAFTSMEN, FOREMEN, & SKILLED LABOR 18.0%
OPERATIVES, SEMISKILLED, & UNSKILLED LABOR 20.5%
OTHER SOCIAL & RECREATIONAL TRAVEL 18.5%
TO & FROM WORK 22.7%
PLEASURE RIDES 14.1%
BUSINESS TRIPS RELATED TO EARNING A LIVING 21.3%
VACATIONS 4.8%
OTHER FAMILY BUSINESS 7.3%
SHOPPING - 6.0%
EDUCATIONAL, CIVIC, & RELIGIOUS 2.8%
MEDICAL & DENTAL 2.5%

Figure 20.—Passenger-car travel classified by major purpose of travel and by occupation of principal operator; summer of 1951, four States.

COMPARISON WITH EARLIER STUDIES

The road-use and motor-vehicle-allocation studies made by the State highway departments between 1935 and 1940 and the motor-vehicle-use studies made since 1950 were all initiated for the purpose of learning more about the characteristics of motor-vehicle ownership and use than was available from other sources. The motor-vehicle-allocation studies were undertaken primarily to obtain information about the characteristics of motor-vehicle ownership, while the road-use studies were designed to obtain information about the characteristics of motor-vehicle use. The present studies are intended to obtain information comparable to that secured in both of the earlier types of studies, and additional information not obtained in either of the previous studies.

There are some important differences in the procedures employed in the three types of studies. All are sample studies in which the aim is to obtain an indication of the character-

Table 20.—Distribution of passenger-car travel classified by purpose of travel and by occupation of principal operator; summer of 1951, four States [1]

Occupation of principal operator	Distribution of passenger-car travel by purpose of travel												
	Total	Earning a living			Family business				Educational, civic, religious	Social and recreational			
		Total	To and from work	Related business	Total	Medical and dental	Shopping	Other		Total	Vacation	Pleasure rides	Other
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Professionals and semiprofessionals	100.0	45.3	20.1	25.2	18.0	1.8	3.7	12.5	5.8	30.9	6.9	8.4	15.6
Proprietors, managers, and officials	100.0	50.0	21.1	28.9	12.3	3.7	3.3	5.3	1.3	36.4	7.9	11.0	17.5
Farmers and farm managers	100.0	42.4	9.4	33.0	26.9	4.6	12.8	9.5	5.2	25.5	.3	8.9	16.3
Store and office clerks, salesmen	100.0	48.3	24.4	23.9	9.7	.9	3.9	4.9	1.6	40.4	3.3	19.3	17.8
Traveling salesmen, agents, etc	100.0	67.3	33.4	33.9	5.3	.1	2.2	3.0	.7	26.7	2.3	13.8	10.6
Craftsmen, foremen, skilled laborers, etc	100.0	45.0	31.9	13.1	11.4	1.9	4.1	5.4	1.3	42.3	4.5	19.3	18.5
Operatives, semiskilled and unskilled workers, and laborers	100.0	41.1	28.4	12.7	13.5	1.3	5.2	7.0	2.4	43.0	6.9	14.8	21.3
Protective services	100.0	58.9	41.5	17.4	8.9	.2	3.8	4.9	.6	31.6	6.5	16.1	9.0
Personal-service workers	100.0	34.1	17.4	16.7	18.2	1.4	11.2	5.6	1.4	46.3	5.5	13.6	27.2
Housewives	100.0	22.1	7.6	14.5	27.9	3.3	15.9	8.7	1.8	48.2	5.5	16.1	26.6
Miscellaneous [2]	100.0	22.0	7.5	14.5	24.5	8.1	5.3	11.1	5.4	48.1	4.8	15.3	28.0
All occupations	100.0	44.0	22.7	21.3	15.8	2.5	6.0	7.3	2.8	37.4	4.8	14.1	18.5

[1] Arkansas, Louisiana, Oklahoma, and Wisconsin.
[2] Includes retired persons, occupations not otherwise classified, and occupations not reported.

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istics of a "universe" by collecting information from a representative sample of cases selected from the universe. In most instances the universe selected for the earlier studies was the motor-vehicle registrations of the last previous complete year. The basic universe selected for the newer motor-vehicle-use studies was the total number of dwelling units in the State being studied, although in some instances (not among the States reported on in this article, however) the commercial-vehicle registrations have been used as a supplemental universe to obtain information on the characteristics of ownership and operation of certain classes of freight-carrying vehicles.

Methods of Sampling

The information sought in the road-use and motor-vehicle-use studies was obtained through personal interviews with the respondents. On the other hand, the information desired for the motor-vehicle-allocation studies was obtained through replies submitted by respondents to mail questionnaires. For the road-use studies, the practice employed in most States was to obtain interviews from a sample of approximately 2 percent of the passenger-car owners and 5 percent of the truck owners. Data on commercial buses were generally obtained on a 100-percent sample basis, with various methods being employed in the different States to collect information about publicly operated buses, such as school buses.

One notable departure from the general methods described above was employed in those States in which the so-called "school method" of obtaining interviews was used. In these States, the study was set up as a school project in certain civics or government classes in selected high schools. Information about the ownership and operation of trucks and buses was usually obtained by regularly employed interviewers.

In contrast to the procedures employed in the motor-vehicle-use studies, where the interviewers are given no latitude in selecting persons to be interviewed, considerable latitude was given to the road-use survey interviewers in selecting their interview prospects. The method most commonly employed involved the assignment of area or place quotas to the interviewers who were instructed to make their contacts on a more or less random basis. Controls were imposed, however, to insure that the samples obtained would conform closely to known conditions in certain respects. These controls included the requirement that urban interviews be distributed among persons employed in various occupations approximately according to the percentage distribution of these workers in the various occupations. Another requirement was that the year-model distribution of vehicles included in the sample be approximately in agreement with the comparable distribution of vehicles registered. A third requirement imposed wherever possible was that interviews taken in unincorporated areas be distributed among residents on the various road systems in proportion to the distribution of dwelling units among the systems as found in the road inventory. In a few instances, interviewers were given lists of persons to be contacted, but since these lists were usually drawn from the registration lists of the previous year the percentage of contact failures was generally high and this method of interviewing proved unduly expensive.

In designing the basic sampling procedures for the motor-vehicle-use studies, advantage was taken of the more scientific sampling procedures that had become general practice during the intervening period. The procedures adopted were essentially those used for the home-interview origin-and-destination studies that had been made by the State highway departments during the last few years in many of the larger cities. The sampling procedures used are described early in this report.

Nature of Travel Information Obtained

Perhaps the most important difference between the road-use surveys and the motor-vehicle-use studies is found in the nature of the travel information obtained. In the earlier studies, an attempt was made to obtain data about the trips made to various points and for various purposes during an entire 12-month period, generally the 12-month period immediately preceding the date of the interview. The basic procedures designed for the present studies call for obtaining data only for trips completed on the most recent weekday and Saturday and Sunday. For the extended or complete motor-vehicle use study these procedures provide that one-fourth of the interviews should be obtained in each area sampled in each of the four seasons of the year. The effect of these changes in procedures should be to reduce memory bias or unreliability in the information obtained, and to provide uniform seasonal coverage of the areas sampled.

In spite of the shortcomings of the procedures employed in the road-use and motor-vehicle-allocation studies, the results obtained in most States were surprisingly satisfactory, insofar as could be learned from comparisons made with known data or estimates calculated independently by other means. Data developed from those studies have been used extensively in many different ways.

Reconciliation of System Classifications

It is not possible to provide direct compa sons of the information obtained in the roa use and motor-vehicle-use studies for vario reasons, one of the most important being t differences in highway system classificati employed for the two studies. Table 21 co pares the mileages of the various system clas fications of highways adopted for the t studies in each of the four States. The syste classifications adopted for the earlier studi were not necessarily the official or legal ones the time the studies were made, but were u classifications made especially for the financi and road-use studies of the highway planni surveys. The system mileages were obtain from tabulations prepared for the financi studies.

For this comparison an attempt was ma to bring the system classifications used in t earlier studies in line with those used in t motor-vehicle-use studies. In the case of Wi consin, which has separate official county a town road systems, treated separately in t road-use studies, all such roads were group into one system, and the data for them co bined for purposes of comparison with t motor-vehicle-use studies. In the case Louisiana, the so-called primary system designated for the road-use study includ most of the mileage treated separately "other State highways" in the motor-vehicl use study, and some mileage which has sin been transferred to parish (county) juri diction.

Comparison of Travel Distributio

Figure 21 presents what may perhaps b the most significant comparisons that can b made between the findings of the road-us and motor-vehicle-use studies. In preparin this presentation, certain combinations o data were made to place the information fro both studies on a comparable basis. It wi be seen that the rural-urban travel distribu tion for residents of unincorporated areas wa very similar in 1951 to that found in th earlier studies. For residents of all place combined, the travel distribution in 1951 wa found to be generally similar to that of th earlier studies, except that the 1951 studie

Table 21.—Comparison of highway system mileages in 1936–37 and 1951; four States

Road system	System mileage							
	Arkansas		Louisiana		Oklahoma		Wisconsin	
	1936	1951	1937	1951	1937	1951	1936	1951
Roads in unincorporated areas:								
State primary system	8,597	9,204	16,771	2,207	7,945	9,647	9,206	9,983
State secondary system	-----	-----	-----	11,794	-----	-----	-----	-----
Local rural roads	45,734	55,417	22,091	25,778	93,244	82,272	72,239	75,894
Subtotal	54,331	64,621	38,862	39,779	101,189	91,919	81,445	85,877
Streets in incorporated places:								
Extensions of State roads	530	512	747	911	492	527	771	1,236
Other streets	4,079	4,034	2,941	4,790	6,240	6,287	7,113	7,674
Subtotal	4,609	4,546	3,688	5,701	6,732	6,814	7,884	8,910
All roads and streets	58,940	69,167	42,550	45,480	107,921	98,733	89,329	94,787

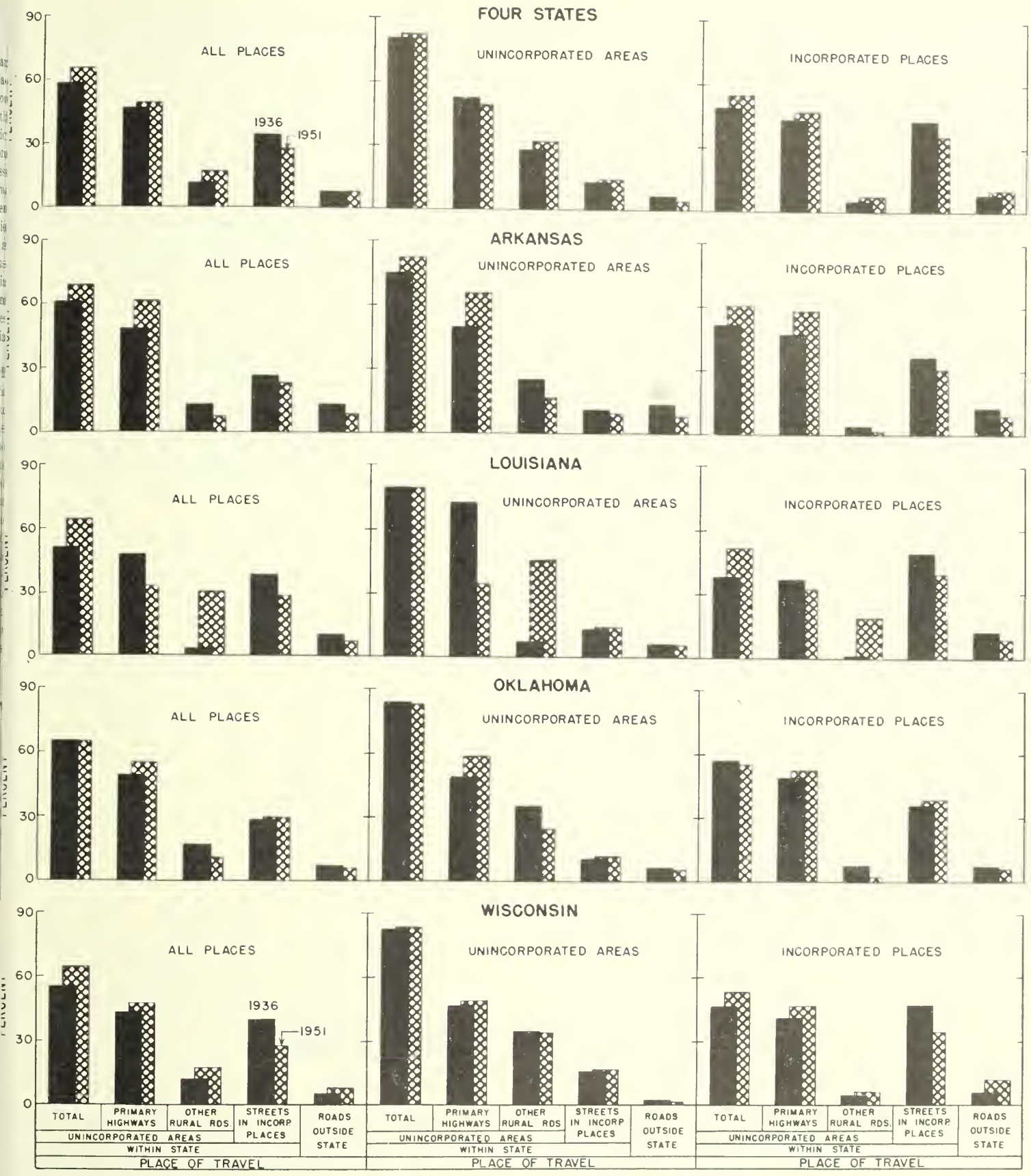

Figure 21.—Comparison of distribution of passenger-car travel by residents of all places, unincorporated areas, and incorporated places, as shown by the 1951 motor-vehicle-use studies and the 1936 road-use studies; four States.

ndicated a somewhat smaller proportion of otal travel was performed on the streets in ncorporated places.

The only important difference noted in rural travel distribution was in the case of Louisiana, but the increase in percentage of total miles driven on "other rural roads" in 1951 as compared with the earlier studies can probably be traced largely to a transfer

of a sizeable block of road mileage from the State primary to the State secondary system and a transfer of some roads to parish (county) jurisdiction.

Another factor affecting the comparisons between the two studies is the change in the rural-urban composition of the population. Oklahoma in 1940 was 38 percent urban and in 1950 was 50 percent urban. Arkansas and Louisiana each reported an increase of about 10 percent in urban population. Wisconsin reported about the same relation between urban and rural population as was found in 1940.[5]

It will be noted that in three of the four States the percentage of travel performed on streets of incorporated places was shown to be less in 1951 than in the earlier studies. This change cannot be explained completely, but there are certain factors which appear to make it seem reasonable. One is that the earlier studies were made during depression years when most people had less money to spend on taking long trips; therefore, a larger percentage of the trips occurred in the cities. In 1951, because of increase in vehicles in use, the travel by all residents of these States was considerably greater, in some instances almost twice as great, than it was at the time of the road-use study. Therefore, the total miles traveled on city streets was more in every case in 1951 than it was in the earlier studies. However, the greater proportionate increase in the miles traveled on rural roads was such as to have the effect of reducing the percentage of total travel on streets of incorporated places.

For all population groups combined in the four States, the percentage of the mileage being driven outside the State of residence was the same for the two studies. The unincorporated areas reported a smaller percentage of out-of-State travel and the incorporated places a larger percentage than was shown in the earlier studies. Among the individual States all but Wisconsin showed a smaller percentage of out-of-State travel in 1951 than was shown in 1936.

A limited comparison of the distribution of travel by purposes can be made between the two studies in the four States. The basic distribution of travel in the road-use studies was between business and pleasure purposes. The former included travel to and from work, related business travel, and travel on family business such as shopping and going to the dentist. The pleasure travel, which in some instances was subdivided between social and recreational purposes, included vacation travel, pleasure rides, and the travel designated as educational, civic, and religious in the 1951 studies. On the basis of this classification, 63 percent of the travel reported in the early studies in the four States was for business purposes, as compared with 60 percent in 1951.

Although it would have been very desirable, it was not possible to make comparisons between the earlier and later studies, of travel classified by trip lengths. The earlier data were analyzed on a basis which involved classification of trips according to trip lengths only for trips extending beyond city limits. Because of the relatively high percentages of trips made which do not go beyond city limits, especially by residents of urban places, and which are not separable from other trips in the tabulations of trip length data required for the motor-vehicle-use studies, it was not possible to make a meaningful comparison between the findings of the two studies in this respect.

[5] *Census of Population: 1950, Volume I, Number of Inhabitants* (table 1 for each State). Bureau of the Census.

Braking Performance of Motor Vehicles

A New Publication of the Bureau of Public Roads

The Bureau of Public Roads recently published a 170-page bulletin, *Braking Performance of Motor Vehicles*, reporting a study of the braking performance of motor vehicles in everyday traffic, of new vehicles in optimum condition, and of used vehicles before and after brake-system maintenance and improvement. The bulletin describes the effect on brake performance of such factors as speed, weight, vehicle type and capacity, and brake type, and reports on the capabilities of brake testing devices. In all, 319 passenger cars, 8 buses, and 1,071 trucks and truck combinations were tested.

The bulletin discusses the general structure and provides the essential characteristics of practical brake regulations, and furnishes the basic data and interpretations thereof required by appropriate authorities in making intelligent selection of values of performance to be used in the body of any brake regulation.

An advisory committee representing industry and government aided the Bureau in the formulation of the program and procedures of the study, and reviewed and approved the analyses and report. More than 50 individual trucking operators cooperated by furnishing vehicles, drivers, and other facilities.

Braking Performance of Motor Vehicles may be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 55 cents a copy.

U. S. GOVERNMENT PRINTING OFFICE: 1954

A list of the more important articles in PUBLIC OADS may be obtained upon request addressed Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

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UBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
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Highway Bridge Location, No. 1486D (1927). 15 cents.
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Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):

1945, 35 cents. 1948, 65 cents. 1951, 60 cents.
1946, 50 cents. 1949, 55 cents. 1952, 75 cents.
1947, 45 cents. 1950 (out of print).

Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (1950). Out of print.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948). 75 cents.
Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25
Motor-Vehicle Traffic Conditions in the United States, House Document No. 462 (1938):

Part 1.—Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2.—Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3.—Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4.—Official Inspection of Vehicles. 10 cents.
Part 5.—Case Histories of Fatal Highway Accidents. 10 cents.
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PUBLICATIONS (Cont'd)

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.
Standard Plans for Highway Bridge Superstructures (1953). $1.25.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.
Uniform Vehicle Code:

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act (1945). 15 cents.
Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act (revised 1952). 15 cents.
Act III.—Uniform Motor-Vehicle Civil Liability Act (1944). 10 cents.
Act IV.—Uniform Motor-Vehicle Safety Responsibility Act (revised 1952). 15 cents.
Act V.—Uniform Act Regulating Traffic on Highways (revised 1952). 20 cents.
Model Traffic Ordinance (revised 1953). 20 cents.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.
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Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1946).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
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Express Highways in the United States: a Bibliography (1945).
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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1954

FOR OFFICIAL USE

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,445	$13,098	$7,342	289.7	$6,150	$2,969	150.8	$46,754	$23,449	535.1	$66,002	$33,760	975.6
Arizona	8,957	7,929	4,773	94.3	658	448	11.0	6,710	4,803	127.6	15,297	10,024	232.9
Arkansas	14,374	7,859	4,686	327.3	6,110	3,047	142.5	15,382	7,593	373.1	29,351	15,326	842.9
California	22,364	30,741	17,036	152.7	18,458	9,848	61.4	113,337	54,470	304.1	162,536	81,354	518.2
Colorado	15,683	9,327	5,303	179.2	1,877	1,045	44.6	17,769	9,734	167.9	28,973	16,082	391.7
Connecticut	18,796	1,476	738	4.9	822	410	1.0	6,635	3,173	8.1	8,933	4,321	14.0
Delaware	4,801	1,459	731	12.2	1,659	832	.5	7,888	4,291	37.1	11,006	5,854	49.8
Florida	14,408	21,801	11,218	232.9	9,298	4,803	145.3	17,654	9,311	283.1	48,753	25,332	661.3
Georgia	20,777	11,991	6,115	258.4	15,088	7,558	311.3	48,934	22,292	642.8	76,013	35,965	1,212.5
Idaho	8,418	4,162	2,660	70.7	3,322	2,116	66.0	14,272	8,909	210.8	21,756	13,685	347.5
Illinois	38,276	36,717	19,741	393.5	13,972	6,998	47.5	78,863	41,527	464.0	129,552	68,266	905.0
Indiana	28,022	33,365	16,691	140.3	14,700	8,442	49.1	29,282	15,023	135.6	77,347	40,156	325.0
Iowa	15,083	17,685	9,784	410.4	5,017	2,605	172.5	21,154	11,787	750.8	43,856	24,176	1,333.7
Kansas	18,249	9,218	4,634	680.4	6,422	3,251	309.7	18,897	9,692	822.6	34,537	17,577	1,812.7
Kentucky	15,552	10,991	5,943	76.6	7,351	3,675	127.3	26,509	13,322	238.2	44,851	22,940	442.1
Louisiana	17,599	11,968	5,972	107.2	5,727	2,863	25.2	35,287	16,434	153.7	52,982	25,269	286.1
Maine	6,849	7,662	4,039	41.9	2,190	1,197	20.9	14,110	7,014	120.0	23,962	12,250	182.8
Maryland	11,007	15,996	8,127	56.4	8,381	3,646	42.5	10,119	5,196	39.8	34,496	16,969	138.7
Massachusetts	18,532	5,494	2,737	18.9	7,356	3,651	4.5	51,809	24,317	43.8	64,659	30,705	67.2
Michigan	29,985	27,249	13,874	357.6	13,132	6,566	174.3	42,881	21,741	335.6	83,262	42,181	867.5
Minnesota	24,402	6,594	3,366	528.7	2,612	1,491	126.3	26,033	13,658	811.9	35,239	18,515	1,466.9
Mississippi	12,965	12,550	6,293	440.2	6,946	3,643	136.9	22,674	11,266	555.2	42,170	21,202	1,132.3
Missouri	22,496	21,643	11,506	849.3	6,074	3,320	167.1	63,096	32,086	765.6	90,813	46,912	1,782.0
Montana	15,895	13,267	7,835	296.6	6,190	3,760	97.5	17,917	11,056	370.1	37,374	22,651	764.2
Nebraska	19,412	21,454	11,967	750.8	3,776	1,968	134.5	20,708	11,065	653.0	45,938	25,000	1,538.3
Nevada	13,152	2,935	2,508	85.1	1,613	1,366	14.0	3,156	2,605	51.8	7,704	6,479	150.9
New Hampshire	6,354	1,990	994	14.4	897	444	6.1	6,289	3,049	30.3	9,176	4,487	50.8
New Jersey	24,703	6,725	3,362	44.6	1,421	711	10.7	20,830	7,823	21.9	28,976	11,896	77.2
New Mexico	7,379	6,783	4,213	195.0	5,858	3,789	68.3	8,150	5,058	172.9	20,791	13,060	436.2
New York	50,947	56,641	30,412	91.5	50,086	25,040	23.8	198,071	92,905	456.7	304,798	148,357	572.0
North Carolina	24,316	15,003	7,361	281.2	8,390	4,072	85.8	37,960	17,695	507.6	61,353	29,128	874.6
North Dakota	12,522	5,581	2,823	750.0	2,219	1,123	228.3	6,810	3,405	511.6	14,610	7,351	1,489.9
Ohio	36,965	21,524	10,921	62.2	14,953	6,278	23.4	74,901	35,680	195.5	111,378	52,879	281.1
Oklahoma	23,444	11,226	6,204	221.7	9,560	5,004	171.4	19,084	10,100	286.4	39,870	21,308	679.5
Oregon	11,788	2,979	1,804	19.2	4,733	2,900	75.0	13,912	8,500	183.1	21,624	13,204	277.3
Pennsylvania	45,028	8,663	4,432	3.1	30,457	15,011	38.5	90,633	44,115	176.0	129,953	63,558	217.6
Rhode Island	5,277	8,758	4,379	28.8	451	225	1.3	8,007	4,005	37.9	17,216	8,609	68.0
South Carolina	13,448	10,525	5,741	165.7	4,474	2,259	102.1	14,978	7,720	276.3	29,977	15,720	544.1
South Dakota	11,525	10,458	5,979	455.7	1,106	619	107.2	7,408	4,250	341.9	18,972	10,848	904.8
Tennessee	23,376	10,825	5,366	305.7	4,626	2,316	107.0	34,494	15,711	344.3	49,945	23,393	757.0
Texas	57,420	3,617	1,854	101.9	14,006	7,443	286.2	72,525	38,720	1,099.8	90,148	48,017	1,487.9
Utah	8,735	3,760	2,799	54.2	1,570	1,176	51.5	6,485	4,983	70.4	11,815	8,958	176.1
Vermont	4,853	2,962	1,491	35.3	124	62	1.4	7,105	3,539	62.4	10,191	5,092	99.1
Virginia	17,392	14,323	7,220	219.1	6,746	3,415	117.0	22,814	10,448	146.6	43,883	21,083	482.7
Washington	13,508	10,505	5,984	100.7	4,404	2,264	57.7	18,873	9,851	129.4	33,782	18,099	287.8
West Virginia	14,638	9,306	4,698	42.3	2,102	1,074	3.4	15,456	7,723	54.6	26,864	13,495	100.3
Wisconsin	20,443	17,989	9,171	199.5	3,049	1,871	47.8	27,936	13,768	293.1	48,974	24,810	540.4
Wyoming	8,528	2,741	1,816	44.1	1,309	844	21.0	6,414	4,004	136.6	10,464	6,664	201.7
Hawaii	5,293	1,227	614	3.2	2,185	1,073	10.4	7,365	3,921	6.2	10,777	5,608	19.8
District of Columbia	6,083	6,160	3,085	7.9	4,632	1,844	1.1	10,688	5,144	.4	21,480	10,073	9.4
Puerto Rico	10,496	4,930	2,390	32.6	1,605	775	7.1	18,953	8,628	50.1	25,488	11,793	89.8
TOTAL	918,960	620,032	330,732	10,335.6	355,864	183,150	4,237.7	1,533,971	766,559	14,593.4	2,509,867	1,280,441	29,166.9

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Contents of this publication may be reprinted. Mention of source is requested.

Continuous Reinforcement in Concrete Pavement—after 15½ years

A Cooperative Investigation by the Indiana State Highway Department and the Bureau of Public Roads

Reported by HARRY D. CASHELL, Highway Physical Research Engineer, Bureau of Public Roads, and WILMER E. TESKE, Research Engineer, Indiana State Highway Department

In 1938 there was constructed near Stilesville, Indiana, as a cooperative research project, a reinforced concrete pavement containing a number of sections, ranging in length from 20 to 1,310 feet. Three types of steel were used, and the amount of longitudinal reinforcement was varied from 0.07 to 1.82 percent among the sections. The purpose of the research was to obtain information on the possibilities for reducing the number of transverse joints in concrete pavement through the use of longitudinal steel reinforcement.

Observation of the experimental pavement sections after 15½ years of service indicates that, with the proper use of longitudinal reinforcement, any spacing of transverse joints within the range studied will give satisfactory performance, without failure of the steel or adverse effects on the concrete.

In the long, heavily reinforced sections numerous closely spaced transverse cracks have developed, but these have not opened and are not detrimental to surface smoothness or pavement life. In fact, an outstanding impression of the pavement is the superficial nature and structurally harmless character of transverse cracks held closed by continuous longitudinal reinforcement. Irrespective of section length, all cracks held closed by reinforcement have been highly resistant to pumping, free of faulting, and have required little or no maintenance.

Longitudinal reinforcement in very large amounts can be used without danger of cracking of the concrete above the steel, buckling failures, or disintegration of the concrete. Longitudinal reinforcement in small amount, 0.11 percent or less, should be used with caution in pavements subjected to heavy truck traffic.

The performance of the longest sections indicates that continuously reinforced pavements will be less susceptible to pumping than pavements of other designs, although under certain conditions of traffic and subgrade they will not be immune from pumping along the longitudinal free edge.

This investigation was planned primarily to determine the extent to which various types and amounts of longitudinal steel reinforcement could be used to reduce the number of transverse joints in concrete pavements. Thus its purpose was to produce information mainly on structural performance. The general excellence of the performance of the pavement sections over a period of more than 15 years raises a question regarding the economic benefits of reinforcing steel that cannot be answered at this time or by this investigation alone. However, subsequently there have been constructed several other continuously reinforced pavements of varying designs which are being observed and reported upon. It is believed that as time develops more information, it may eventually become possible to evaluate the relative economics of continuously reinforced pavement and that of more usual designs.

THIS is the fifth of a series of reports of a cooperative research investigation undertaken in 1938 by the Indiana State Highway Department and the Bureau of Public Roads to study the effects of varying amounts of continuous longitudinal steel reinforcement in portland cement concrete pavement sections of various lengths. Preceding reports (*1–4*)[1] have described in detail the design and construction of the project and the performance and behavior of the sections during the first 10 years of service. The present report describes the condition of the various sections as observed during a performance survey conducted in the spring of 1954 when the pavement was 15½ years old. On the assumption that the reader is either familiar with this investigation or can easily obtain previously published information about it, only those data essential for clarification are repeated in this report.

The experimental reinforced sections, ranging in length from 20 to 1,310 feet, were constructed near Stilesville, about 30 miles west of Indianapolis, Ind., as part of the eastbound lanes of the divided highway U. S. 40. The continuous steel reinforcement included rail-steel bars, billet-steel bars, and wire fabric, of varying sizes and weights such as to provide percentages of longitudinal steel ranging from 0.07 to 1.82 (see table 1).

In addition to the regular sections listed in table 1, four special 500-foot sections were constructed in which weakened-plane joints were spaced at 10-foot intervals and the bond between the longitudinal steel and the concrete was broken purposely for a distance of 18 inches on each side of each joint.

In compliance with 1938 standard specifications, the pavement is 20 feet wide with a 9–7–9-inch thickened-edge type cross section. Construction was directly over a natural fine-grained soil (table 2) common to the Indianapolis area and now known to be highly susceptible to pumping.

Traffic flow (fig. 1) on U. S. 40 in the immediate vicinity of the experimental sections has increased steadily since 1938, reaching in 1953 an estimated annual average daily volume (in both directions) of nearly 8,000 vehicles, trucks and buses comprising approximately 25 percent of the total.

Scope of Performance Survey

The 15½-year performance survey of the experimental pavement consisted primarily of a critical examination of the various sections for possible inherent characteristics and weaknesses that might affect or shorten their service life. During the survey attention was particularly focused on: (1) The effect of transverse cracking on surface deterioration and concrete durability; (2) the manner in which the various types and quantities of longitudinal steel performed their structural function of holding cracks tightly and permanently closed; (3) the susceptibility of the sections to pumping and the influence of pumping at the joints, cracks, and edges of the pavement; (4) the maintenance require-

[1] Numbers in parentheses refer to the bibliography, p. 141.

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ments of the sections; (5) the effect of traffic on pavement behavior and condition as indicated by a comparison between the heavily traveled outer lane and the more lightly traveled passing or inner lane; and (6) the surface smoothness of the sections.

Conclusions

Constructed on a pumping-susceptible soil and subjected to relatively heavy traffic for 15½ years, the experimental sections have by their behavior and condition provided basic information applicable to the ultimate performance of reinforced concrete pavement. The most significant conclusions to be drawn from the results of this investigation follow:

Regular sections

1. Sections of any length can be reinforced with continuous longitudinal steel in sufficient amount to maintain all transverse cracks in a tightly closed condition without adverse effect on the concrete.

2. The length of a continuously reinforced section containing adequate steel has a pronounced effect on the pattern of transverse crack development. In sections up to approximately 150 feet in length the cracks will be comparatively few and variably spaced. In the end regions of longer sections the same condition will exist while in the central region of long sections numerous closely spaced cracks will form.

3. The rate of crack development and the crack frequency will have little effect on the service life of adequately reinforced sections.

4. Surface widths of cracks held closed by continuous reinforcement will increase gradually with time by raveling and wearing superficially under exposure and the action of traffic; however, this condition will not impair the durability of the concrete and will require little maintenance.

5. Cracks held closed by continuous reinforcement will be highly resistant to pumping, practically free of faulting, and will seldom develop surface roughness.

6. Welded wire fabric as light as 32 pounds per 100 square feet and longitudinal bar reinforcement as light as ¼-inch bars spaced at 6-inch centers appear to be structurally incapable of performing the intended function of holding transverse cracks permanently closed in pavement sections subjected to heavy traffic.

7. A continuously reinforced concrete pavement without joints will be less susceptible to the damaging effects of pumping than rigid pavements of usual current designs but, under certain conditions of subgrade type and traffic, will not be immune from pumping action at the longitudinal free edge of the pavement.

Special sections

1. Preservation of pavement continuity is an essential requirement of the design comprising short, constructed slabs held together by relatively lightweight reinforcement.

2. To preserve continuity of the pavement surface, longitudinal reinforcement heavier than that provided by wire fabric of 91 pounds per 100 square feet would be required if shear bars are omitted at the weakened-plane joints. If shear bars are included in the design, however, pavement continuity can be preserved with an amount of longitudinal reinforcement somewhat less than that provided by the 91-pound fabric.

3. As long as continuity exists, the pavement will be free from all but very slight faulting at the weakened-plane joints, of acceptable surface smoothness, and highly resistant to pumping.

4. The submerged type, weakened-plane joint with a copper seal enveloping the bottom parting strip appears to provide more protection against the entrance of water to the subgrade than that obtained with the surface-

Table 1.—Details of reinforcement in the experimental sections

Number of sections for each length [1]	Length of each section [2]	Calculated maximum stress in steel [3]	Weight of reinforcement	Reinforcement size and spacing				Percentage of longitudinal steel [5]	Average tensile strength of longitudinal steel	
				Longitudinal		Transverse				
				Diameter [4]	Spacing c. to c.	Diameter [4]	Spacing c. to c.		Yield point	Ultimate
	Feet	*Lb. per sq. in.*	*Lb. per 100 sq. ft.*		*Inches*		*Inches*	*Percent*	*Lb. per sq. in.*	*Lb. per sq. in.*
Rail-Steel Bars (Deformed)										
2	600	25,000	567	1-in.	6	½-in.	24	1.82	63,000	113,200
	840	35,000								
	1,080	45,000								
	1,320	55,000								
4	340	25,000	334	¾-in.	6	½-in.	24	1.02	64,400	113,300
	470	35,000								
	610	45,000								
	740	55,000								
4	150	25,000	167	½-in.	6	½-in.	24	.45	68,800	115,300
	210	35,000								
	270	45,000								
	330	55,000								
6	80	25,000	94	⅜-in.	6	⅜-in.	24	.26	66,700	93,600
	120	35,000								
	150	45,000								
	180	55,000								
6	40	25,000	50	¼-in.	6	¼-in.	12	.11	60,300	84,600
	50	35,000								
	60	45,000								
	80	55,000								
Billet-Steel Bars (Deformed), Intermediate Grade										
2	360	15,000	567	1-in.	6	½-in.	24	1.82	46,900	78,000
	600	25,000								
	840	35,000								
	1,080	45,000								
4	200	15,000	334	¾-in.	6	½-in.	24	1.02	49,100	78,500
	340	25,000								
	470	35,000								
	610	45,000								
4	90	15,000	167	½-in.	6	½-in.	24	.45	51,400	78,600
	150	25,000								
	210	35,000								
	270	45,000								
6	50	15,000	94	⅜-in.	6	⅜-in.	24	.26	55,500	81,900
	80	25,000								
	120	35,000								
	150	45,000								
6	20	15,000	50	¼-in.	6	¼-in.	12	.11	56,900	77,300
	40	25,000								
	50	35,000								
	60	45,000								
Wire Fabric (Cold-Drawn Wires)										
6	140	25,000	149	No. 0000	4	No. 3	12	0.42	.....	81,800
	190	35,000								
	250	45,000								
	310	55,000								
6	90	25,000	107	No. 0000	6	No. 3	12	.28	.....	80,300
	130	35,000								
	170	45,000								
	200	55,000								
6	80	25,000	91	No. 000	6	No. 4	12	.24	.....	89,100
	110	35,000								
	140	45,000								
	170	55,000								
6	60	25,000	65	No. 0	6	No. 6	12	.17	.....	83,700
	80	35,000								
	100	45,000								
	120	55,000								
6	30	25,000	45	No. 3	6	No. 6	12	.11	.....	81,000
	50	35,000								
	60	45,000								
	80	55,000								
6	20	25,000	32	No. 6	6	No. 6	12	.07	.....	88,700
	30	35,000								
	40	45,000								
	50	55,000								

[1] The term "section" as used in this report refers to a lane or 10-foot width of pavement; thus the numher "2" indicates a pair of sections, one being on each side of the center joint.

[2] The lengths of the longer sections are nominal lengths and may he either 5 or 10 feet greater than the actual length in cases where a pair of hridge-type joints were installed.

[3] Calculated by subgrade resistance method with coefficient of resistance equal to 1½ times the weight of the pavement.

[4] The rail-steel and billet-steel reinforcement were round hars. The diameters of the wires in the fabric were as follows: No. 0000, 0.3938 inch; No. 000, 0.3625 inch; No. 0, 0.3065 inch; No. 3, 0.2437 inch; No. 4, 0.2253 inch; No. 6, 0.1920 inch.

[5] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab.

Table 2.—Subgrade soil data [1]

Total soil samples for each depth	Location below surface	Material retained on No. 270 sieve	Plasticity index	Moisture content
	Inches	*Percent*	*Percent*	*Percent*
SECTIONS CONTAINING RAIL-STEEL BARS				
18	0–4	19	12	13.2
	4–12	19	13	16.1
	12–24	21	13	18.1
SECTIONS CONTAINING BILLET-STEEL BARS				
15	0–4	24	11	12.6
	4–12	23	12	15.6
	12–24	24	12	18.1
SECTIONS CONTAINING WELDED-WIRE FABRIC				
12	0–4	36	10	11.2
	4–12	33	12	13.4
	12–24	32	13	14.1

[1] Average data from tests on samples taken from the finished subgrade just prior to placing concrete.

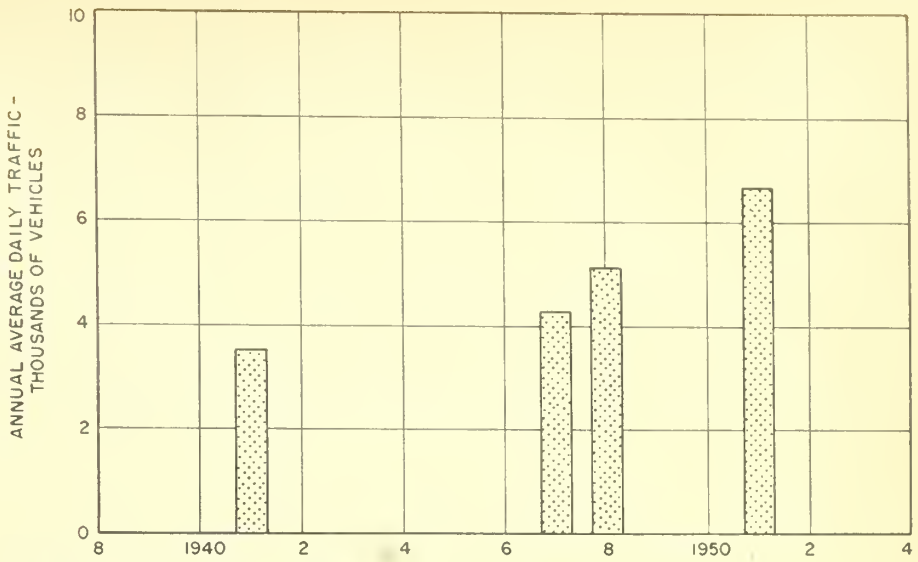


Figure 1.—Traffic volume on U. S. 40 in the immediate vicinity of the experimental pavement.

groove type of joint. However, the fractures that form above the parting strips will be meandering in character and unsightly in appearance.

5. Elimination of expansion joints and the consequent minimizing of progressive opening of the weakened-plane joints would be advantageous to pavements of this design.

Development of Cracking

Crack development during the first 10 years of pavement life has been carefully recorded and fully described in preceding published reports; consequently, only data indicative of the general trend of cracking are reported here.

Typical examples of crack patterns that formed during the 15½-year period of service are shown in figure 2. To emphasize the more recent fractures, only those that appeared within the final time period covering the last 5½ years are labeled with their survey number (7).

The data of figure 2, although limited, give a good idea of the crack development existing in the range of section lengths comprising the experimental pavement. Briefly, the shorter sections are comparatively free of fractures. As the length of section increases, however, cracking is more frequent until in the central region of the longest sections the crack interval is often less than 2 feet.

Figures 3 and 4 show the development of cracking over the 15½-year period in two selected cases, a 310-foot section and the critical central region of the 1,310-foot section. In this report, unless otherwise noted, fractures less than 5 feet long and those that occurred at the extreme ends of sections as a result of pumping at the joints are not considered in the various analyses of cracking.

Figure 3 indicates the manner in which the fractures appeared with respect to time. As will be noted, the rate of cracking was very high during the first year following construction, after which the rate was comparatively low, remaining rather uniform for the intermediate-length section and diminishing progressively with time in the case of the central area of the long section. Of special interest is the fact that 13 percent of the cracks now present in the central 400 feet of the 1,310-foot section formed during the last 5½ years. This suggests a continued high stress condition in the concrete in that region, with continued bond between the steel and the concrete.

Figure 4 shows the effect of pavement age on the average distance between cracks. Of interest is the average crack interval of 2.3 feet (range from 1 to 6 feet) now existent in the central portion of the 1,310-foot section reinforced with 1.82 percent longitudinal steel.

In the discussion of figures 3 and 4 attention has been directed purposely to crack development in the central portion of the 1,310-foot section, because it is indicated that cracking

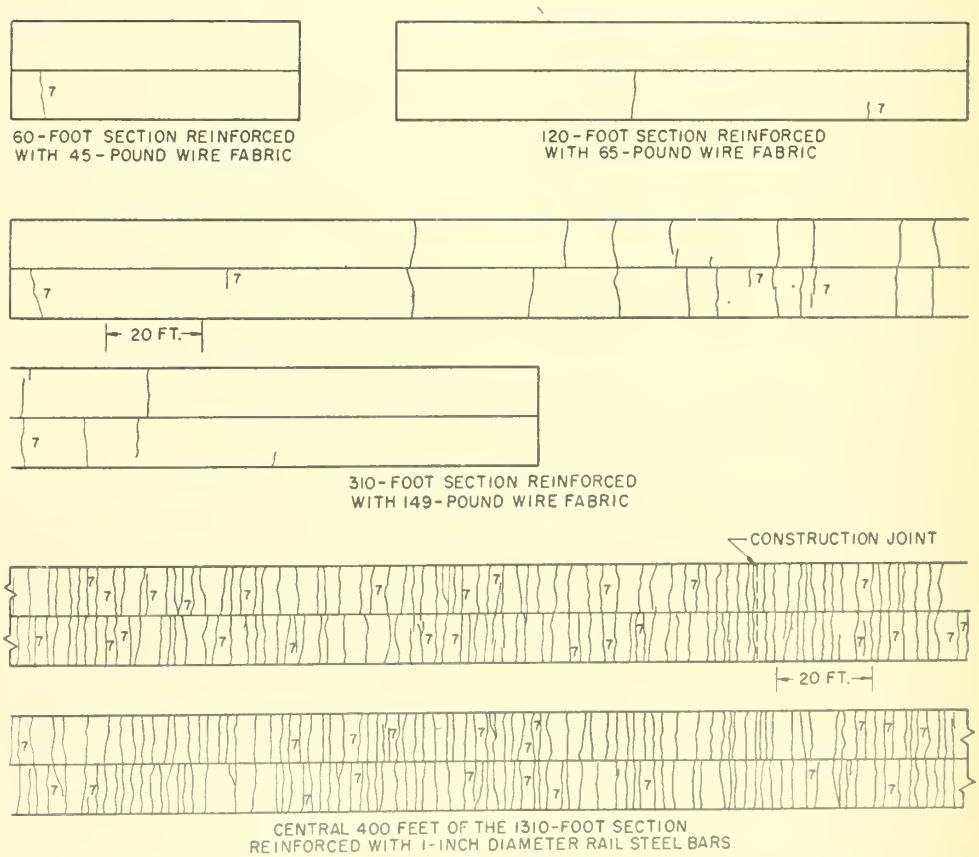

60-FOOT SECTION REINFORCED WITH 45-POUND WIRE FABRIC
120-FOOT SECTION REINFORCED WITH 65-POUND WIRE FABRIC
20 FT.
310-FOOT SECTION REINFORCED WITH 149-POUND WIRE FABRIC
CONSTRUCTION JOINT
20 FT.
CENTRAL 400 FEET OF THE 1310-FOOT SECTION REINFORCED WITH 1-INCH DIAMETER RAIL STEEL BARS

Figure 2.—Typical crack patterns in sections of various lengths after 15½ years. (Fractures numbered 7 occurred between the sixth and seventh crack surveys, October 1948 and April 1954.)

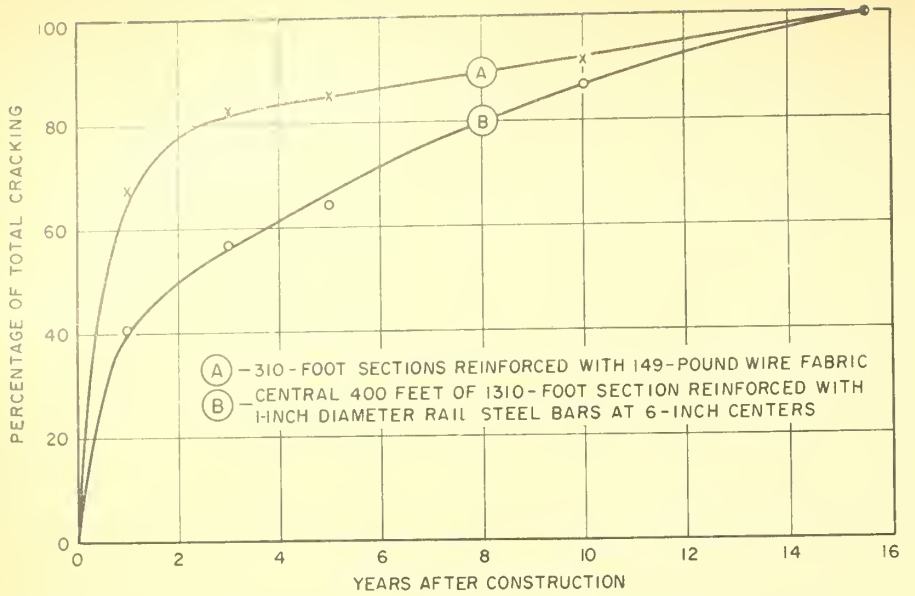


Figure 3.—Rate of crack development during the first 15½ years of pavement life.

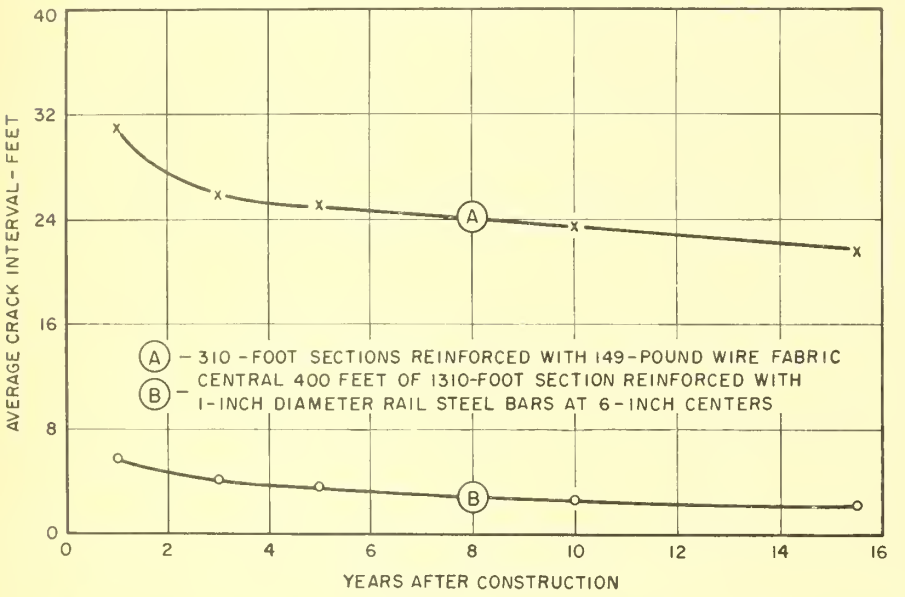


Figure 4.—Relation between average transverse crack interval and age of pavement.

in this area approximates closely that which would develop in the critical area of a pavement of infinite length continuously reinforced with 1.82 percent steel. Other, more recent researches (*5–10*) have shown, however, that continuous pavement without joints can be reinforced adequately with considerably less longitudinal steel than the 1.82 percent used in the 1,310-foot sections. Comparison between the results of the later researches and the one under discussion indicates that cracks will form at somewhat greater intervals in the critical region of continuous pavement containing lesser (and hence more economical) amounts of longitudinal steel than in the central portion of the heavily reinforced 1,310-foot section. This is as might be expected.

Discussion of cracking has thus far been confined largely to that which develops in continuously reinforced sections of considerable length. An analysis of the crack data obtained from shorter sections reinforced with various weights of welded wire fabric is shown in figure 5. In this figure the average crack interval or slab length, at pavement age of 15½ years, is plotted against section length as constructed. Slab length is defined as the distance between two expansion joints, two transverse cracks or an expansion joint and a crack.

The curves of figure 5, despite limitations, suggest a general pattern of cracking for reinforced sections up to approximately 250 feet in length; that is, the average slab length increases with an increase in section length until a peak value is reached, beyond which there is a rapid decrease in the average length of slabs. For conditions existing in the sections under discussion, the longest average slab lengths are found at a section length of about 130 feet.

It is indicated that, for weights of wire fabric of 91 pounds per 100 square feet or less, the relation between section length and average slab length is unaffected by the amount of reinforcement present. For greater weights of reinforcement, however, there appears to be a pronounced effect. For example, in the case of a 140-foot section, the average slab length was 70 feet for welded fabric ranging in weight from 45 to 91 pounds per 100 square feet, whereas the average slab length for the 107- and 149-pound fabric was 86 and 120 feet, respectively.

Character and Effect of Cracking

An outstanding impression of this and all previous condition surveys is the superficial nature and structurally harmless character of transverse cracks held closed by continuous longitudinal steel. The vast number of fractures that occurred in the longer, more heavily reinforced sections, however shocking when considered numerically, have not weakened materially the structural continuity of the pavement. Irrespective of crack incidence or rate of crack development, all sections, except those reinforced with the lightest amount of each type of steel, have continued in service free from distress at the cracks other than inconsequential edge raveling and very limited damage caused by localized scaling and spalling.

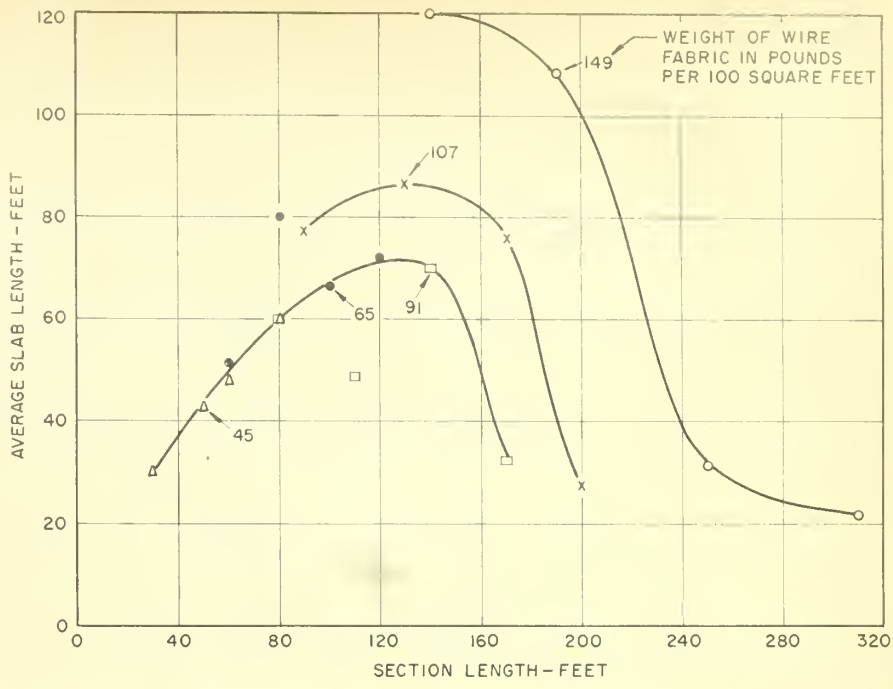
Over the years, exposure and the action of traffic have produced various degrees of raveling and wearing of the surface edges of the transverse cracks. At present, especially in the long, heavily reinforced sections, a few of the earlier and many of the more recently formed fractures are almost imperceptible at the surface of the pavement, being discernible only by extremely close inspection. In contrast, other fractures in the same sections and of the same age group are quite conspicuous at the surface of the pavement, having raveled and rounded to widths which, at the surface, average approximately one-eighth inch.

Cracks in the lightly reinforced short and moderate-length sections are, in general, more readily seen than those in the more heavily reinforced longer sections and, for a given long section, cracks in the central region are more prominent than those in the end regions.

Figure 6 shows the surface condition of the pavement at several transverse cracks as observed in the outside wheel path of the heavily traveled lane. These photographs, taken in 1954, are of fractures that occurred at an early age in the central portions of the longest sections reinforced with 1-inch diameter rail-steel bars, ½-inch diameter rail-steel bars, 149-pound wire fabric, and 91-pound wire fabric, the percentages of longitudinal reinforcement being 1.82, 0.45, 0.42, and 0.24, respectively. By selection, the surface widths of the fractures shown are representative of the widest resulting from raveling that were observed in each respective group of sections.

The cracks shown in figure 6, although restricted in length, serve to illustrate several conditions or characteristics of cracking that

Figure 5 (**Right**).—*Relations between weight of wire fabric, section length, and average crack interval or slab length.*



were quite obvious when actually seen in the pavement, namely:

1. The surface width of an individual crack is variable along its length.

2. The surface widths of cracks in the shorter, more lightly reinforced sections are wider, on the average, than those of cracks in the heavily reinforced longer sections.

3. The extent of raveling or wearing now present at the edges of cracks held closed by continuous longitudinal reinforcement can hardly be considered as detrimental to surface smoothness or to pavement life.

Another very important structural characteristic existing at all cracks held closed by continuous reinforcement, but not apparent in figure 6, is the fineness of the real widths of the cracks themselves; that is, widths which, unlike those observed at the surface, are unaffected by raveling and wearing of the crack edges. The real widths of all cracks are many times smaller than their respective surface widths. Like their surface widths, the real widths increase with a decrease in the percentage of longitudinal reinforcement, other conditions being equal.

The erratic surface raveling of the cracks throughout their lengths naturally makes it impossible to establish exact values of their surface widths. Nevertheless, to disclose behavior trends, if any, a procedure of measurement was adopted which gives estimated values of the average surface widths of cracks. Essentially, this procedure consisted of dividing a crack extending the width of a lane into three equal segments, then making a width measurement in each segment at a point judged to be average. The value obtained by averaging the three measurements is considered to represent the average surface width of the crack for the particular lane. Although this method is rather crude, it is believed that the averages of a number of such measured values have significance in relative comparisons.

At pavement ages of 10 and 15½ years, measured values of the average surface widths of numerous cracks were obtained in the manner described. Figure 7 shows, for each of the two ages, the relation established by plotting values obtained in the heavily traveled lane against corresponding percentages of longitudinal reinforcement. Each plotted value represents an average of a number of fractures that formed at an early age in the central area of the longest section of a group of similarly reinforced sections.

Comparisons available in figure 7 indicate that the surface widths of cracks tend to decrease with an increase in the amount of longitudinal steel in spite of the fact that the heavier the reinforcement the longer the length of section. As expected, the surface widths of cracks tend to increase with pavement age. It is of interest to note, however, that during the last 5½ years the rate of surface raveling

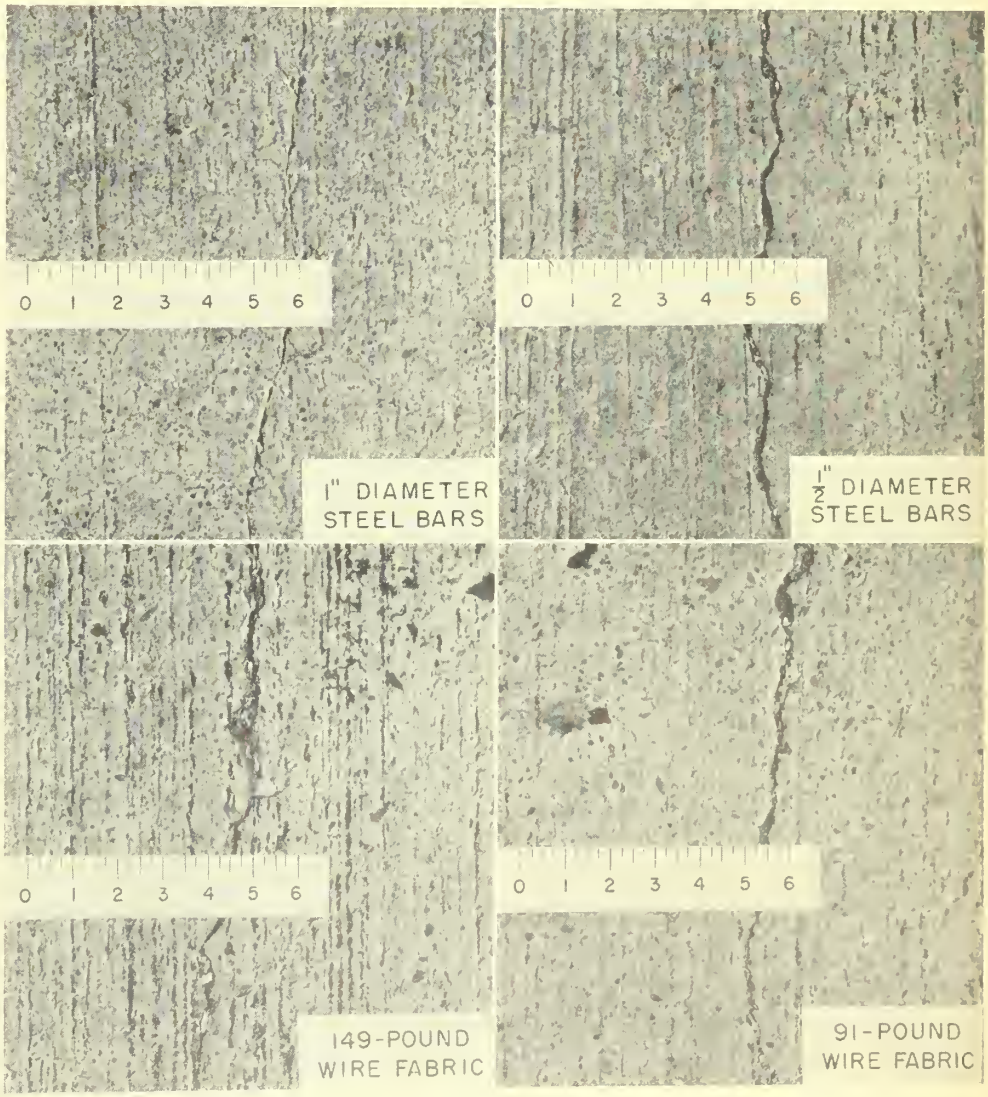


Figure 6.—Surface condition at cracks typical of those that developed at an early age in the central portion of the longest section of each group of similarly reinforced sections (these were in the heavily traveled lane, after 15½ years of service).

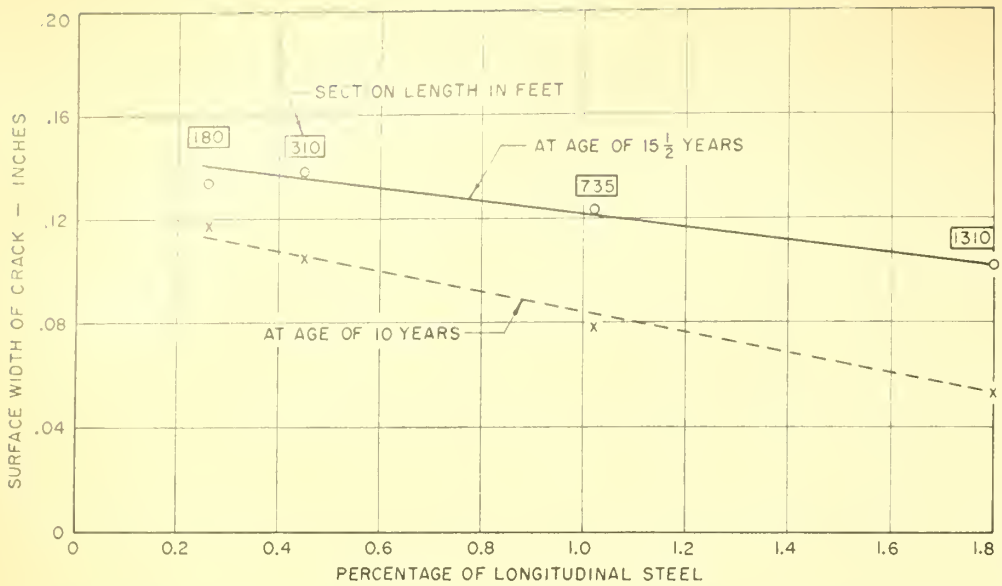


Figure 7.—Effect of percentage of longitudinal steel and pavement age on the surface widths of cracks. The values shown are of fractures that occurred at an early age in the central portion of sections in the heavily traveled lane.

of the selected cracks in the more heavily reinforced sections has been higher than that of the selected cracks in the shorter sections containing the lighter reinforcement. This trend is the opposite of earlier developments but, at present, seems to have little significance.

Damage and Maintenance of Cracks

Concrete deterioration, other than the natural raveling and wearing just reported, was extremely rare at all cracks held closed by continuous reinforcement. Of the few cases of more than superficial damage observed at the cracks, the worst is pictured in figure 8. This type of damage, classified as surface spalling, was first noticed during the 15½-year performance survey.

Surface scaling is another type of pavement distress that was first observed during the last survey. Currently, the scaling is confined to scattered pavement areas, is light in depth, and appears to be more prevalent in the immediate vicinity of cracks and joints than elsewhere.

With reference to scaling, it should be mentioned that during the last four winters calcium chloride, as an additive to sand or cinders for ice control, has been applied to the experimental pavement in indeterminate amounts whenever conditions warranted its use. The light scale observed in 1954 may have been caused by this treatment. Whether or not the many transverse cracks will accelerate its development can be, for the present, only a matter of speculation.

With the exception of isolated cases of spalling, such as that shown in figure 8, it is the opinion of all those who have examined critically the more heavily reinforced sections that the inconsequential surface widths and the fineness of the transverse cracks themselves definitely preclude the need for bituminous maintenance. Opinion has been divided, however, with respect to maintenance of the closed cracks in the sections containing the more conventional amounts of reinforcement.

Performance and Limitation of Reinforcement

For many years the required amount of longitudinal steel reinforcement for pavement sections has been computed by an analytical method based on the possible maximum tensile stress that could be induced in the steel by subgrade resistance during contraction of fractured sections. In the application of this method to the design of the experimental sections it was assumed that subgrade resistance would be constant and could be expressed as a coefficient equal to 1½ times the weight of the pavement. On this basis, the section lengths given in table 1 would develop the listed calculated maximum stresses in the steel, and the yield-point values that were anticipated for each of the three types of reinforcement would have been approached closely in the longest section of each group. This was expected to produce, under repeated stressing, inelastic elongation and possible breakage of the steel with a consequent opening of the cracks.

However, the performance survey at the end of 15½ years showed that all percentages of longitudinal steel of 0.17 and greater had been able to maintain in a closed condition all cracks in their respective group of sections. It is apparent, therefore, that the design assumptions of the experimental sections were too conservative to determine, as was intended, the limiting length of section for each of the selected steel sizes shown in table 1. Comparisons were further complicated by the fact that tests on the steels used showed that, in general, the yield-point values were considerably higher than had been anticipated.

As observed in table 1, the longitudinal steel incorporated in the various sections ranged from 0.07 to 1.82 percent, or from a minimum equivalent to the lightest used in pavement design to a maximum exceeding any other known pavement installation.

After 15½ years of service the performance of the sections containing the maximum percentage of steel indicated definitely that longitudinal reinforcement, when positioned near the mid-depth of the pavement slab, can be used in very large amounts without danger of cracking above the members, failures caused by buckling, or disintegration of the surrounding concrete.

Figure 8.—Extreme case of spalling at a crack in a section reinforced with heavy steel bars. Even minor spalling at such cracks is unusual.

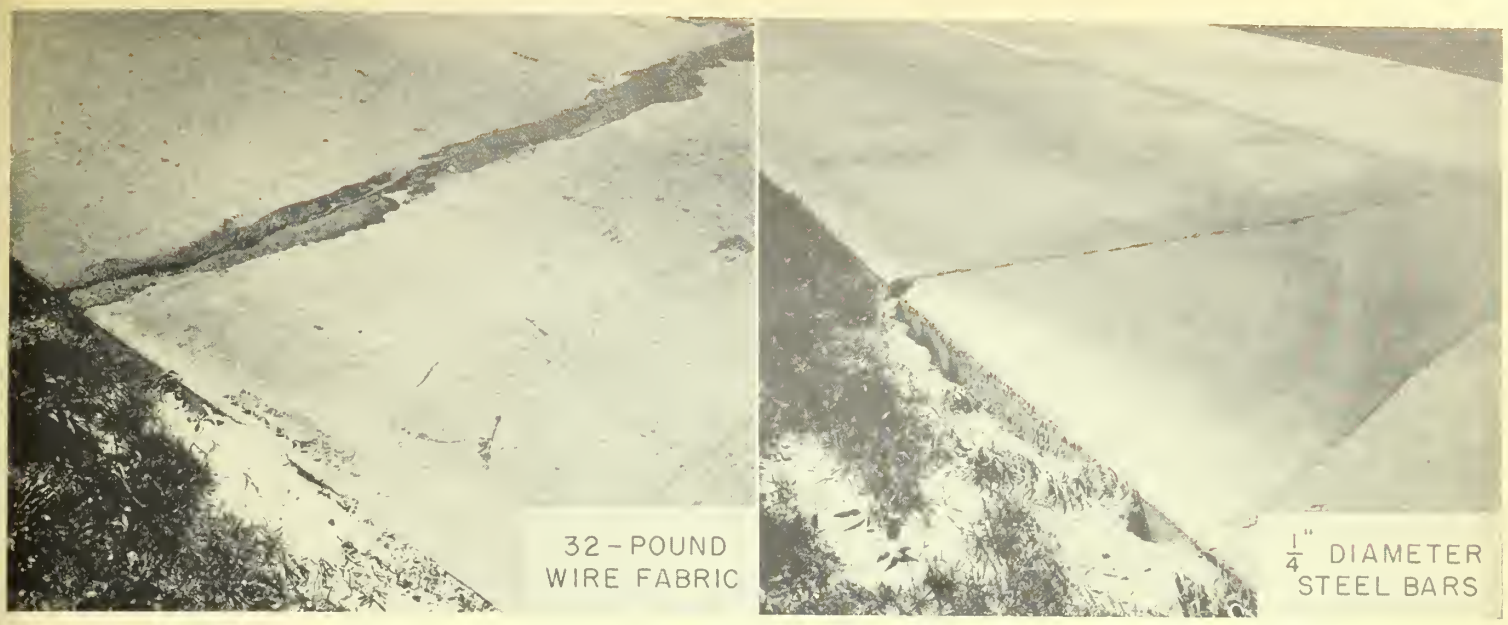


Figure 9.—Cracks at which the reinforcing steel failed. Steel failures in the heavily traveled lane were generally followed by pumping and faulting.

As implied earlier, a number of open cracks, indicative of steel failures, developed in sections containing the lighter percentages of longitudinal steel (0.11 and 0.07 percent). Figure 9 shows two of these failures that appeared in the heavily traveled lane sometime between the 10-year and 15½-year performance surveys. In this lane, pumping and faulting developed rapidly after the cracks opened and bituminous maintenance was soon needed to alleviate surface roughness.

The development of steel failures in the sections containing longitudinal reinforcement of 0.11 percent or less is shown in table 3. Of interest is, first, the length of time required for the steel to rupture after a crack had formed and, second, the fact that section length exerted little influence on the occurrence of the ruptures.

As indicated in table 3, steel failures were first observed during the 8-year performance survey. At that time three breaks were noted in the heavily traveled lane, all at cracks in sections reinforced with the 32-pound wire fabric (0.07 percent longitudinal steel). Although two of these cracks were present at pavement age of 1 year and the other at 5 years the steel did not fail until sometime between the fifth and eighth years. At the end of 15½ years of service, seven additional steel failures were observed in the sections containing the 32-pound wire fabric, four at cracks in the heavily traveled lane and three at cracks in the passing lane. Two of the cracks in the heavily traveled lane were present at pavement age of 5 years and the other two at 10 years, whereas two of the cracks in the passing lane were first seen at the end of 1 year and the other at 10 years. Of the cracks at which steel breakage was found, three were in 30-, three in 40-, and four in 50-foot sections.

The steel failures that occurred at cracks in sections reinforced with ¼-inch diameter rail- and billet-steel bars (0.11 percent longitudinal steel) took even longer to develop than those in sections containing the 32-pound wire fabric.

All of the five failures in these sections were

Table 3.—Steel failures at cracks in the experimental sections

Failure location			Year after construction [1] when—	
Section length	Pavement lane		Crack first observed	Steel failure first observed
Feet	Outer	Inner		
Sections Containing 32-Pound Welded-Wire Fabric				
30	X		5	8
30	X		10	15½
30		X	10	15½
40	X		1	8
40	X		5	15½
40		X	1	15½
50	X		1	8
50	X		5	15½
50	X		10	15½
50		X	1	15½
Sections Containing ¼-Inch-Diameter Billet-Steel Bars				
20	X		3	15½
40	X		10	15½
Sections Containing ¼-Inch-Diameter Rail-Steel Bars				
60	X		1	15½
80	X		5	15½
80		X	5	15½

[1] Performance surveys were made shortly after construction and at the end of 1, 3, 5, 8, 10, and 15½ years.

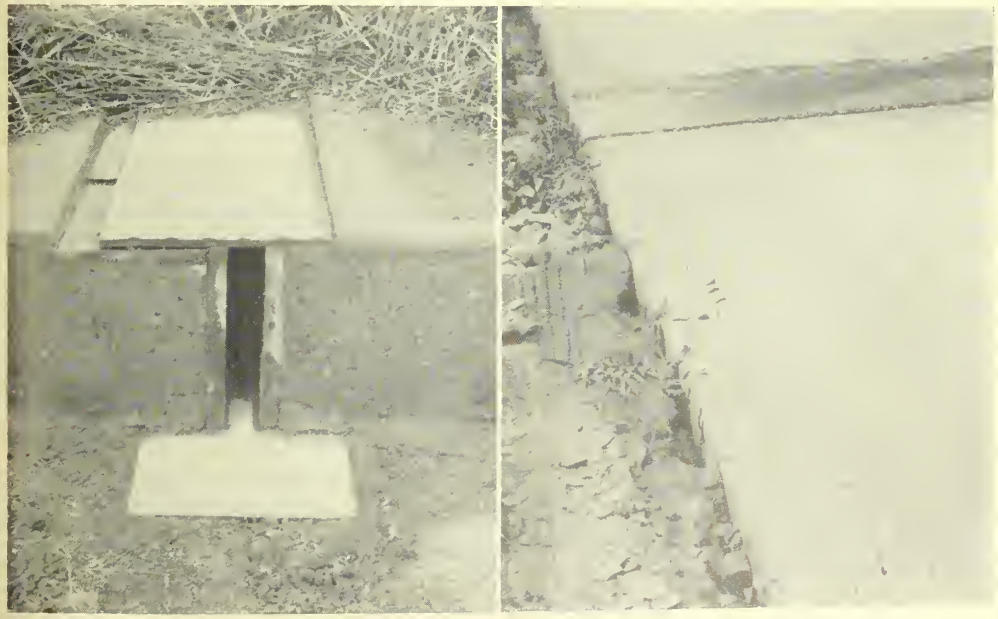


Figure 10.—The bridge-type joint, and early pumping at such a joint.

Figure 11.—Condition after 15½ years of a double bridge-type expansion joint, typical of the majority. Loosening of the steel cover plate, pumping, and faulting developed at an early age at most of these joints in the heavily traveled lane.

first observed during the 15½-year performance survey. Four were at cracks in the heavily traveled lane and one at a crack in the passing lane. One of these cracks was present at pavement age of 1 year, one at 3, two at 5, and the fifth at 10 years. Of the five steel failures (three for rail steel and two for billet steel), one each occurred in the group of 20-, 40-, and 60-foot sections and two occurred in the group of 80-foot sections.

The crack survey data provide a comparison between the behavior of the longitudinal steel in the sections containing the 32-pound wire fabric and the longitudinal steel in the sections containing the slightly heavier ¼-inch diameter rail- and billet-steel bars. At the time of the 5-year survey, six cracks had formed in the heavily traveled lane of the sections reinforced with the 32-pound wire fabric and nine such cracks had formed in the sections reinforced with the ¼-inch diameter steel bars. The 15½-year performance survey indicated that the reinforcement had failed at five of the six cracks referred to in the sections reinforced with the wire fabric while, in contrast, the reinforcement had failed at only three of the nine cracks that had occurred during the first 5 years in the sections reinforced with the steel bars. This difference in performance is probably a reflection of the difference in the percentage of longitudinal steel in the two groups of sections.

It is evident from table 1 that sections reinforced with the 45-pound wire fabric contain the same percentage of longitudinal steel as those reinforced with the ¼-inch diameter bars. At the end of 15½ years only four cracks were observed in the sections containing the 45-pound wire fabric. Crack survey data disclose that one of these was first seen at the end of 5 years, two at 10 years, and the fourth at 15½ years. All were in closed condition.

The exact cause or causes of the steel failures which have been described are not known. However, their occurrence irrespective of section length and their predominance in the heavily traveled lane, despite tie-bars in the longitudinal joint, point to repeated stresses induced by traffic loads as a major factor in their development. The nature of the failures suggests the need for extreme caution and a consideration of the possible effects of live-load stresses in the use of longitudinal reinforcement as light as 0.11 percent.

Development and Effect of Pumping

By the end of 15½ years, pumping had affected various parts of the experimental pavement. In the following discussion of this development it will be of interest to keep in mind the steady increase in traffic volume recorded during the life of the experimental sections (fig. 1).

Bridge-type expansion joints

Early in the life of the pavement, pumping began to appear at some of the bridge-type expansion joints (fig. 10) which were designed to provide the wider openings between the longer sections. These joints had no effective means of load transfer, except the steel cover plate, and consequently, under traffic, permitted slab deflections considerably larger than those occurring at any other part of the constructed pavement. After 5 years of service many of the joints of this type were pumping, faulting was noticeable, and flexural cracking, caused by loss of subgrade support through pumping, had occurred in the end areas of several sections. At 15½ years, nearly all of the bridge-type joints in the heavily traveled lane were subject to pumping and, at many, flexural cracking, serious settlement, and severe faulting of the slab ends had developed. Of all companion joints in the passing lane, however, only three were affected by pumping.

Figure 11 illustrates typically the general condition of the pavement existing at the majority of the double bridge-type expansion joints. When the photograph was taken, in the spring of 1954, most of the double and many of the single bridge-type joints were sources of acute surface roughness in spite of rather extensive maintenance. At 15½ years these joints had reached the stage where major repairs were badly needed and such repairs were on immediate schedule. Although the action of pumping contributed materially to the poor condition of these joints, it must be recognized that all joints, regardless of type, which separate long sections, undergo large changes in width and, even when pumping is controlled, are susceptible to functional defects which contribute to surface roughness.

Doweled expansion joints

Seven years after construction, incipient pumping appeared at several of the conventional doweled expansion joints which separate the shorter sections. The provision for load transfer in the joints consisted of ¾-inch diameter dowels spaced at 12-inch centers. After 10 years, pumping was observed at a number of these joints, but the action was still so slight that faulting was negligible and flexural cracking of the slab ends had not occurred. At the end of 15½ years, however, the performance survey showed that pumping at the doweled joints in the heavily traveled lane had progressed to the point of causing considerable structural damage. Evidence of pumping was observed at more than half of the joints, approximately one-third had pumped to the extent that flexural cracking had developed in the slab ends, and faulting, as much as ½ inch, was present at the outside corners of several. In contrast, pumping was completely absent and faulting was negligible at the same type of joint in the passing lane.

In concluding the discussion of pumping and expansion joints, it is of special interest to note that a comparison of the described service performances of the bridge-type joints and the doweled joints indicates that load-transfer devices have considerable value in resisting and delaying the action of pumping. It is also of interest that all percentages of longitudinal reinforcing steel of 0.11 or greater have, thus far, held closed all fractures that occurred, as a result of pumping, in the pavement areas adjacent to the various expansion joints.

Construction joints

Construction joints constitute another structural feature that has been affected by pumping. Essentially, two types of construction joints were used in the experimental pavement: one type in the sections reinforced with the welded wire fabric; the other in the sections reinforced with the deformed bars. In the first, the wire fabric ended at the header board and ⅝-inch-diameter deformed tie-bars,

4 feet long and spaced at 2-foot centers, were installed to hold the adjoining slabs together. In the second, the bar reinforcement was extended through the header board.

Of 11 construction joints of the first type, the 15½-year survey of the heavily traveled lane showed that (1) five joints were subject to pumping, one to the extent of flexural cracking, (2) widths ranged from tightly closed to ¾ inch, (3) faulting had developed at nearly all, the maximum being ¾ inch, and (4) moderate surface spalling of the edges requiring some maintenance was present at the majority. In the passing lane pumping had not appeared at any of the joints, but widths were consistent with those in the heavily traveled lane and slight faulting and minor spalling had developed. Figure 12 shows the condition at one of these joints which, as may be observed, had opened appreciably. Apparently the tie-bars crossing this type of joint did not develop sufficient bond or lacked the strength to perform properly their intended function of maintaining adjoining slabs in close contact.

Under the same conditions 22 of the 25 construction joints of the second type—that is, those containing continuous reinforcement—were found to be structurally sound, tightly closed, and free from pumping. The condition of the pavement at one of these joints, installed in the central portion of a section reinforced with 1-inch diameter bars, is shown in figure 13.

Of the three unsound construction joints of the second type, one each was located in sections reinforced with ¼-, ⅜-, and ½-inch diameter billet-steel bars. All three of the joints were open ⅜ to ½ inch, indicating failure of the reinforcing steel crossing them and, in the heavily traveled lane, all had pumped and faulted. These joint failures occurred sometime after the tenth year of pavement life. Quite perplexing is the failure that occurred at the construction joint in the section reinforced with the ½-inch diameter bars (0.45 percent longitudinal steel). This joint was located only 69 feet from the end of a 270-foot section.

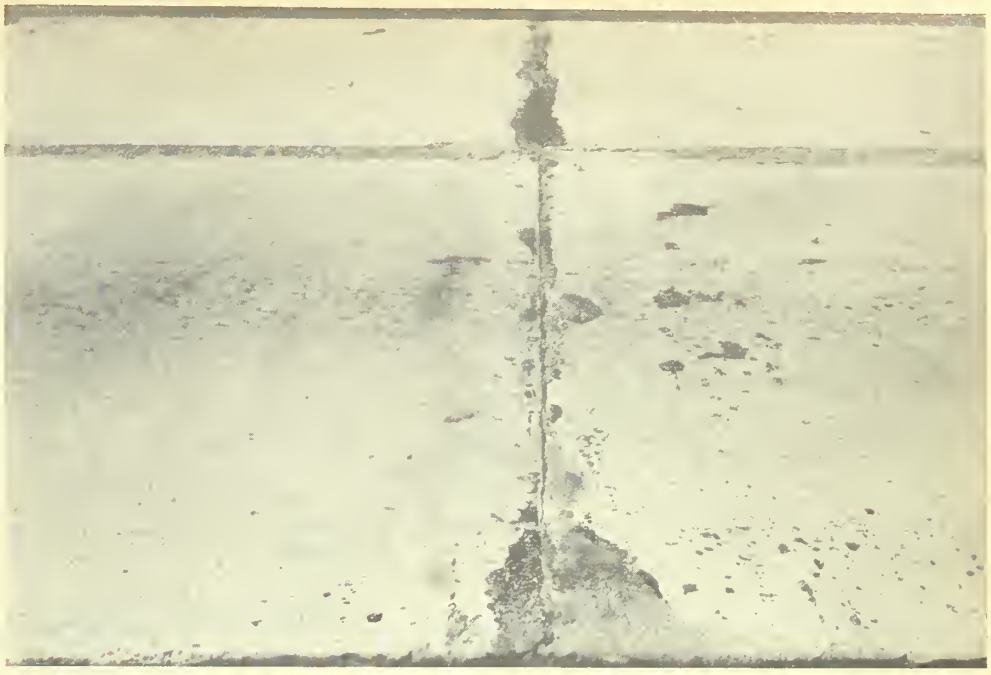


Figure 12.—Condition after 15½ years of a construction joint in sections reinforced with wire fabric. Tie bars were used to hold adjoining slabs together.

Inspection of the pavement in the vicinity of all construction joints through which the reinforcement was continuous showed that these joints had little effect on the crack pattern normal to that part of the section in which they were located. This behavior differs from the abnormal concentration of cracking which has been observed in the concrete placed immediately ahead of the joint in question when construction was resumed on certain other continuously reinforced pavements (*5–10*). Since there is evidence that the method of curing might have exerted some influence on the excessive cracking reported in the pavement adjacent to the construction joints of the other researches, it appears desirable to mention that the pavement under discussion was cured initially with wet burlap and then wet straw for a total of 7 days.

Figure 13.—Condition after 15½ years of a construction joint in the central portion of a section reinforced with heavy steel bars. Reinforcement was continuous through the joint.

Transverse cracks

Pumping in various stages was noted at a number of the previously reported open cracks that formed in the sections reinforced with the lighter amounts of steel. Specifically, the 15½-year survey showed that of 11 such cracks in the heavily traveled lane, 9 were subject to pumping, flexural cracking of the slab ends had occurred at 4, and, in some cases, faulting was quite pronounced. Of four open cracks in the passing lane, slight evidence of pumping was observed at one. Two examples of the condition described may be seen in figure 9.

The appearance of pumping at the cracks shortly after the reinforcing steel ruptured is indicative of the structural weakness resulting from this type of failure and emphasizes the importance of maintaining in closed condition all cracks in reinforced concrete sections.

In contrast to the action at open cracks, neither pumping nor faulting has developed at any of the large number of transverse cracks held closed by continous reinforcement. Considering the prevalence of pumping and faulting elsewhere in the pavement, this per-

Figure 14.—Condition after 15½ years: Slab failure in heavily traveled lane near center of 830-foot section reinforced with 1-inch rail-steel bars. Faulting at longitudinal joint and edge pumping preceded structural breakdown.

formance indicates the watertight character and the inherent structural strength of cracks maintained in closed condition by reinforcing steel.

Longitudinal free edge

Pumping along the free edge of the pavement some distance from the ends of a section was first observed during the 10-year performance survey. At that time the only area affected was in the heavily traveled lane near the middle of an 830-foot section reinforced with 1-inch-diameter bars. In the immediate vicinity of pumping, abnormal raveling or chipping was noted at the surface edges of three closely spaced transverse cracks, indicating that the pavement at that point had been deflected considerably by heavy loads. The underlying cause of the excessive deflection is pointed out by notes taken during construction which describe the subgrade at this point as soft, spongy, and unstable to such an extent that the side forms had to be supported by mudsills of heavy planking during placement of the concrete.

Under continued pumping and deflection the narrow segments of the pavement within the boundaries of the three transverse cracks soon shattered and the broken pieces settled and faulted. In spite of considerable surface maintenance this failure became progressively worse and at the time of the last survey had developed to the condition shown in figure 14. As indicated in the photograph, the adjacent pavement of the passing lane is entirely free from similar distress.

To date, the described failure is the only one of its nature to appear in any of the experimental sections. During the last survey, however, several areas of potential trouble were noted in the same section in which the failure developed. The worst of these is shown in figure 15. Typical of the areas were an open and slightly faulted longitudinal joint, indicating slight settlement of the heavily traveled lane, and an excessive raveling or chipping of the edges of transverse cracks, indicating abnormal and localized deflection of the pavement under heavy loads. Except for a complete absence of pumping, these areas resemble closely the early stages of the failure just reported.

At the time of the 15½-year survey, evidence of pumping was observed at four additional locations along the free edge of the pavement, all being in the heavily traveled lane of relatively long sections. The action, however, had been very slight and the pavement in the areas was by no means in immediate structural jeopardy. Figure 16 shows the most advanced stage of edge pumping observed at any of the locations. An exceptionally low shoulder existed at all four of the pumping locations and three of the four, like that of the photograph, were within 100 feet of the ends of long sections where transverse cracking was minimum.

Effect of Traffic

Throughout the report, reference has been made repeatedly to either the heavily traveled or the passing lane. Consequently, it is probably needless to recall that the experimental two-lane pavement is one-half of a rural, divided highway; and, typically, the outer lane carries the greater number of vehicles and practically all of the heavy trucks, the inner lane being used largely for passing. Such a distribution of traffic naturally provides an excellent opportunity to observe the effect of repetition of heavy loads on the structural performance and condition of the various sections.

Development of cracking

The 10-year crack survey showed that 53 percent of the transverse cracks then present in both lanes of the pavement had formed in

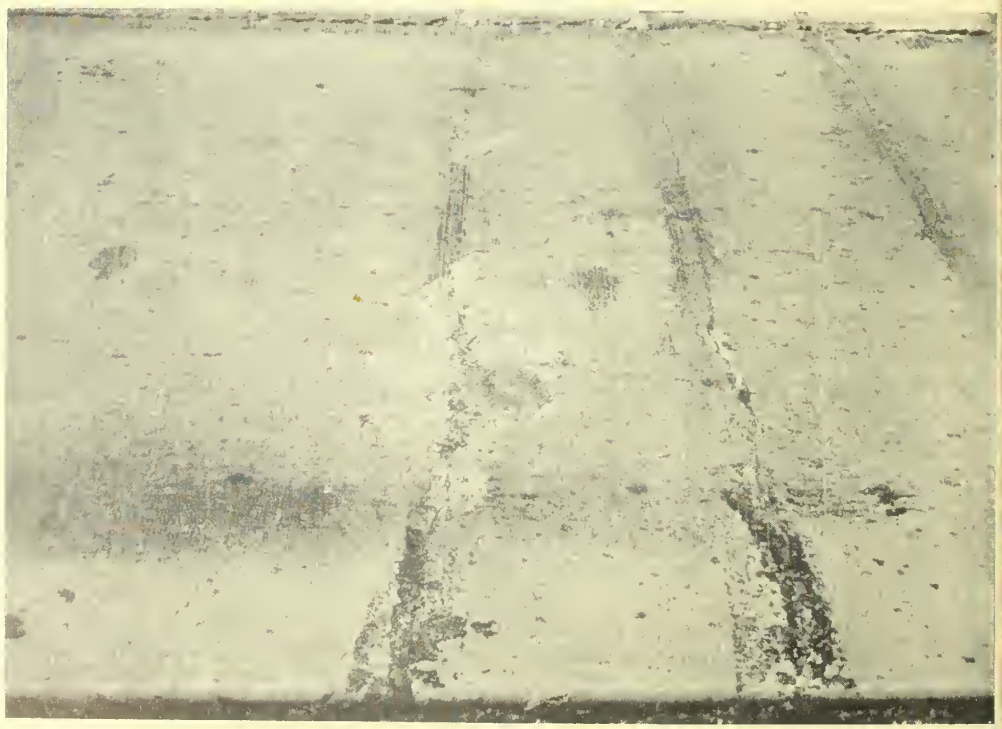


Figure 15.—Condition after 15½ years: Potential failure in the central portion of 830-foot section reinforced with 1-inch rail-steel bars. Slight faulting at the longitudinal joint indicates settlement of the heavily traveled lane. This development is approximately 50 feet east of the failed area shown in figure 14.

he heavily traveled outer lane. A similar urvey at the end of 15½ years indicated hat (1) of the total number of cracks in the entral area of the 1,310-foot section, only 51 ercent were in the heavily traveled lane nd (2) of the total number of cracks (those aused by pumping omitted) in all sections of 00 feet and less, 60 percent were in the eavily traveled lane. Thus it appears that epetition of traffic loads has exerted some nfluence on the development of transverse racks, particularly in the shorter sections. n the restrained, multifractured central egions of long sections the influence is very light, however.

Surface widths of cracks

In all of the later crack surveys it has been uite apparent that the greater volume of raffic using the outer lane causes considerably nore raveling and wearing of the edges of the ransverse cracks than the lighter traffic on he passing lane. From data obtained at the nd of 15½ years, figure 17 was prepared to ompare the effect of traffic on the surface widths of cracks measured in the two lanes of the pavement. As indicated by the figure, he difference between lanes is appreciable, he average surface widths of cracks in the eavily traveled lane being approximately hree times those of companion cracks in the assing lane. In addition to the described ifference in crack raveling, all spalling, such as that shown in figure 8, is confined to the eavily traveled lane.

Pumping

Since one of the conditions necessary for the development of pumping is the repetition of heavy loads, it is natural to expect that evidence of pumping would be most pronounced in the lane of the experimental pavement carrying the greater number of heavy trucks. A review of reported data discloses that rather serious pumping developed at a considerable number of transverse joints and open cracks in the heavily traveled lane in contrast to only four cases of slight pumping action at such points in the passing lane. From this it might be concluded that the need for provisions to prevent or control pumping is greater for the heavily traveled lanes of divided highways than for the passing lanes.

The difference in the present condition of the two lanes of the experimental pavement indicates an inherent difference in the structural requirements of the inner and outer lanes of the pavement on divided highways and suggests the possibility of differences in design to compensate for the characteristic distribution of traffic on that type of pavement.

Pavement Smoothness

Since the advent of the automobile the general public has been deeply interested in the riding quality of the roads over which it travels. Each vehicle affords a means for evaluating and comparing the smoothness or roughness of pavements and every motorist has become a critic of riding comfort. As a result, a continuous smooth riding surface is a requisite of modern highways and, within

Figure 16.—Early stage of edge pumping observed at pavement age of 15½ years. This condition developed over a 50-foot length near the east end of a long, heavily reinforced section. The arrows point to tiny pump holes.

economic limits, the common goal of all pavement design.

During the last survey, motoring at legal speed over the experimental pavement resulted in the following impressions: Starting in the heavily traveled lane and at the beginning of a long section, the ride was pleasantly smooth with only an occasional faint sensation of dipping. At the first double bridge-type joint, however, an abrupt jolt or shock accompanied by vehicle vibrations was felt. Smooth riding continued again until another joint of similar type was encountered. Passing on to the intermediate-length sections separated by the single bridge-type joints, the impressions just described were repeated but on a more moderate scale, the effect of joint roughness being less but at closer intervals. Continuing to shorter sections and the conventional expansion joints, a slight bump and pitching motion was felt at some of the joints, and at nearly all of them an annoying clicking sound was heard, suggesting that the sealing material was no longer flush with the surface of the pavement. Travel over the very short, lightly reinforced sections containing open cracks was accompanied by a succession of light bumps and slight pitching.

In motoring over the passing lane, it was noted that disturbances at most joints were far less severe than those encountered at adjacent joints in the heavily traveled lane. In contrast, however, with the exception of damaged areas, there was no impression of a

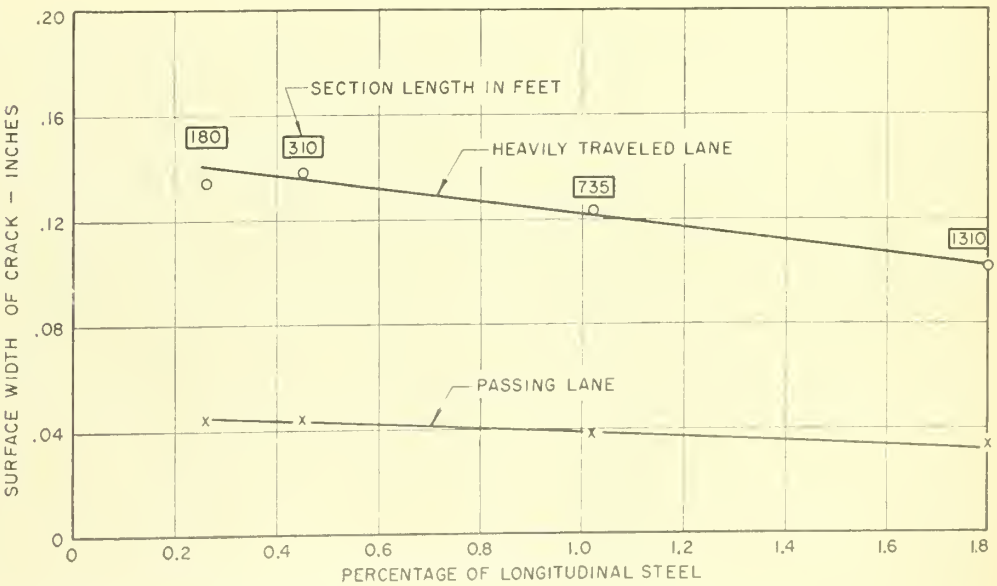


Figure 17.—Effect of traffic on the surface widths of cracks at pavement age of 15½ years.

Table 1.—Comparison of the roughness indexes of the various groups of similarly reinforced sections with the average index of the pavement between joints of the long, heavily reinforced sections [1]

Section length Group average	Percentage of longitudinal joints in section	Ratio of roughness index of a given group of sections to that of pavement between joints of long sections	
		Outer lane	Inner lane
Feet	*Percent*		
Sections Containing Rail-Steel Bars			
960	1.82	1.1:1	1.0:1
540	1.02	1.1:1	1.0:1
240	.45	1.2:1	1.0:1
133	.26	1.0:1	1.0:1
58	.11	1.3:1	0.9:1
Sections Containing Billet-Steel Bars			
720	1.82	1.1:1	1.0:1
405	1.02	1.1:1	0.9:1
180	.45	1.2:1	1.0:1
100	.26	1.2:1	1.0:1
43	.11	1.3:1	1.0:1
Sections Containing Welded-Wire Fabric			
223	0.42	1.2:1	1.0:1
148	.28	1.1:1	1.0:1
125	.24	1.1:1	1.2:1
90	.17	1.1:1	1.2:1
55	.11	1.1:1	1.0:1
35	.07	1.4:1	1.4:1

[1] All roughness indexes were determined at pavement age of 15½ years.

difference between lanes in the riding quality of the pavement other than at the joints.

From the motorist's viewpoint the long stretches of pavement between the expansion joints of the heavily reinforced sections left little to be desired in riding comfort, indicating that the many cracks that formed in these sections have not affected the surface smoothness of the pavement.

Another, more analytical approach to the relative riding quality of the various sections was provided by the road surface roughness indicator developed some years ago by the Bureau of Public Roads (*11*). With this device it is possible to compare the surface roughness of sections or groups of sections by means of an index expressed in inches per mile.

At the end of 15½ years, roughness indexes were obtained for each group of similarly reinforced sections listed in table 1. In addition, a weighted average index was established for the pavement located between the joints of the long, heavily reinforced sections by excluding 50 feet of pavement on each side of the transverse joints separating these sections. This index should approximate closely that which would have been obtained had these sections been constructed as continuously reinforced pavement without joints.

The results of the 15½-year roughness survey are given in table 4. In this table the indexes determined for each of the various groups of similarly reinforced sections are compared with the index representative of continuous pavement without joints. The section length listed in the first column of the table is the average of the range in section lengths included in each group of sections and indicates the average spacing of transverse joints.

It is apparent in table 4 that, in spite of a rather narrow range in the surface roughness of the various groups of sections, certain trends exist which support the impressions previously described. For example, in the outer, heavily traveled lane, the roughest pavement surface is shown for the groups of very short sections reinforced with the lightest amount of each type of steel. As discussed earlier, open cracks developed in the sections of these groups. Also, in the same lane, the ratios for the groups of long, heavily reinforced sections reflect the roughness existing at the bridge-type joints; whereas those for the groups of shorter sections indicate that the surface of the basic pavement

Table 5.—Distinguishing features of the four 500-foot special sections [1]

Length of section	Spacing of weakened-plane joints	Type of weakened-plane joint	Weight of reinforcement
Feet	*Feet*		*Lb. per 100 sq. ft.*
500	10	Surface-groove	91
500	10	do	45
500	10	Submerged	91
500	10	do	45

[1] Bond between the steel and concrete was purposely destroyed for 18 inches on each side of each joint. Shear bars consisting of ¾-inch diameter dowels, 18 inches long spaced at 12-inch centers, were placed in one-half of each of the four sections. Copper seals enveloped the bottom parting strips of the submerged type, weakened-plane joints

with its large number of transverse cracks is somewhat smoother than that of pavements containing conventional expansion joints. In the inner or passing lane, the majority of the groups of sections have the same surface smoothness as that of the basic pavement, suggesting that, under certain conditions, pavements with expansion joints can retain surface smoothness equal to those of continuously reinforced pavements without joints.

Another interesting relation disclosed by the roughness indexes determined at the end of 15½ years, but not apparent in table 4, is the fact that the pavement located between the joints of the long sections in the heavily traveled lane was of approximately the same smoothness as that of companion pavements in the passing lane, in spite of the fact that the surface widths of the cracks in the heavily traveled lane were, on the average, three times those in the passing lane.

Behavior of Special Sections

It may be recalled that four special 500-foot sections containing weakened-plane joints at 10-foot intervals were included in the experimental pavement to develop information on the practicability of a design in which trans

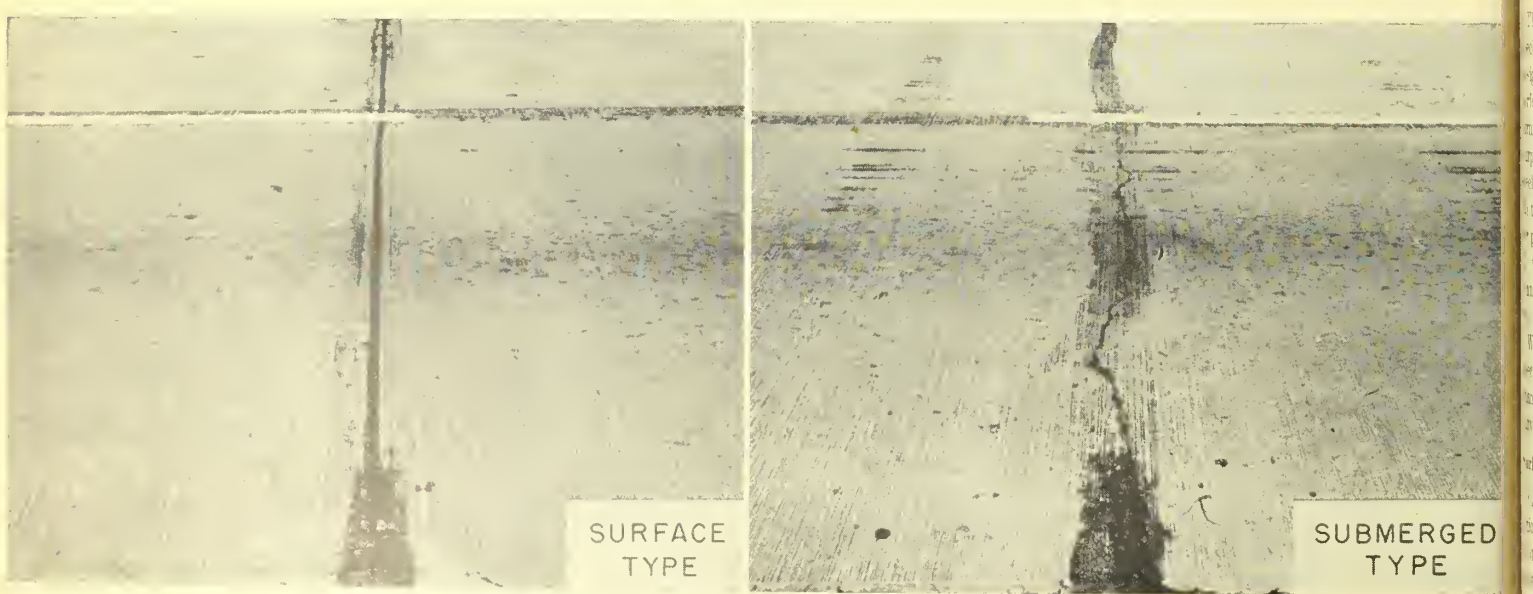


Figure 18.—Condition after 15½ years of weakened-plane joints, typical of those containing shear bars and continuous, structurally sound 91-pound wire fabric.

Figure 19.—Condition after 15½ years of weakened-plane joints, typical of those without shear bars and at which the reinforcing steel failed.

verse crack control is effected through the use of short constructed slabs with pavement continuity provided by continuous, relatively lightweight, longitudinal reinforcement. As explained in earlier reports, the basic feature of the proposed design is the deliberate breaking of the bond between the steel and the concrete for a distance of 18 inches on each side of each joint. Through this feature the reinforcing steel can elongate appreciably at each joint without exceeding its yield point, making it possible, during contraction periods, to provide a certain degree of relief from tensile stresses induced by subgrade resistance. For example, during a sudden drop in pavement temperature when subgrade resistance is relatively great, the reinforcing steel can elongate and permit the slab units to contract about their individual centers. Then, subsequently, when the resistance subsides, the steel can contract and draw the units together.

The distinguishing features of the four special 500-foot sections, together with general design details, are given in table 5. The performance of these sections was expected to furnish information on (1) the overall effect of purposely destroying the bond between the steel and the concrete in the manner described, (2) the amount of longitudinal steel necessary for proper functioning of the proposed design, (3) the benefit of shear units in the weakened-plane joints, and (4) the relative value of the two types of weakened-plane joints.

With these objectives in mind, the four special sections were subjected to a close examination during the 15½-year performance survey.

Surface condition of joints

Figure 18 shows the 15½-year surface condition of the two types of weakened-plane joints installed in the special sections. The photographs are of joints in the heavily traveled lane and the condition illustrated is typical of that now present at joints containing shear bars and unbroken 91-pound wire fabric.

A well-sealed joint of good appearance is typical of the weakened-plane joint of the surface-groove type. All structurally sound joints of this type have continued in excellent condition and have required little maintenance. Their behavior indicates that, if pavement continuity is maintained, the comparatively small length changes of the 10-foot slabs are conducive to well-sealed conditions.

Likewise, except where steel failures have occurred, all weakened-plane joints of the submerged type, although rather unsightly in appearance, have continued to remain in excellent condition structurally. The fractures that formed above the parting strips of these joints are, in general, irregular and meandering in character, and, under exposure and the action of traffic, they have raveled and chipped to widths wider than those that developed in the regular sections reinforced with comparable percentages of steel. For example, at the end of 15½ years, the average measured surface width of fractures at structurally sound submerged joints containing the 91-pound fabric was approximately 0.3 inch in the heavily traveled lane or about twice the average surface width of fractures in the same lane of equivalently reinforced, regular sections.

In contrast to the favorable condition found at the joints which were structurally sound figure 19 shows the present state of surface deterioration typical of that existing in the heavily traveled lane at joints, constructed without shear bars, at which the longitudinal steel was either broken or stressed beyond the yield point. Wherever the steel failed at the weakened-plane joints, relatively wide separations (up to ⅜ inch) quickly developed, and raveling and spalling increased appreciably.

Steel failures

The results of that part of the 15½-year examination relating to failures of the longitudinal steel crossing the weakened-plane joints are given in table 6. The data of this table indicate that, in the heavily traveled lane, the reinforcement was either broken or stressed beyond the yield point at 40 of the 196 weakened-plane joints incorporated in the four special sections. Of the 40 steel failures, 37 occurred at joints without shear units (dowels) indicating, rather definitely, that shear forces caused by loads passing over the joints were primarily responsible for the failure. Apparently, the elongation of the 36-inch length of unbonded steel at each weakened-plane joint permitted a separation of the slab ends sufficient to destroy or reduce whatever shear resistance aggregate interlock may have provided and to transfer all or part of the function of resisting shear to the relatively small members of the welded wire fabric.

Considering only the 37 failures that occurred at the joints without shear units, it is noted that 28 appeared in sections reinforced with the 45-pound and 9 in those reinforced with the 91-pound fabric. This

Table 6.—Steel failures and pumping at joints in the heavily traveled lane of the special sections [1] at the end of 15½ years

Weight of reinforcement *Lb. per 100 sq. ft.*	Shear bars	Number of joints with steel failures	Number of pumping joints
SURFACE-TYPE JOINT			
91	Yes	0	0
91	No	7	8
45	Yes	0	2
45	No	15	12
SUBMERGED-TYPE JOINT			
91	Yes	0	0
91	No	2	0
45	Yes	3	0
45	No	13	2

[1] Table gives values for half sections. Either 24 or 25 joints in each half section.

shows that although the 91-pound fabric provided better shear resistance than the 45-pound fabric the No. 500 wires of the heavier fabric were not adequate to provide the shear resistance necessary for good structural performance under conditions such as obtained in these tests.

Of interest is the fact that all of the three steel failures which occurred at the doweled joints in the heavily traveled lane were in the half of the section containing the 45-pound wire fabric and the submerged type, weakened-plane joints. In accounting for these failures, it is possible for a progressive separation to develop at a joint and cause a residual tensile stress in the reinforcing steel. When this happens the relatively lightweight reinforcement, as used in the 500-foot special sections, may be subjected, during contraction periods, to tensile stresses sufficiently large to cause failures. In this connection, data presented in the 10-year report indicated that of the two types of weakened-plane joints, the submerged type developed the greater residual openings.

From a consideration of the causes of the steel failures that have been described it appears that elimination of expansion joints would be advantageous to pavements of the design of the special sections. This would control the progressive opening of the weakened-plane joints and minimize shear failures by maintenance of more positive aggregate interlock.

Pumping and faulting

The distribution of pumping joints in the heavily traveled lane of the special sections is shown in the last column of table 6. At the end of 15½ years, evidence of pumping was observed at nearly all of the impaired joints of the surface-groove type but, conversely, except in two cases, there was little evidence of this action at those of the submerged type. When joints of the submerged type were installed, a copper seal which enveloped the bottom parting strip was incorporated in their design. It may be that these seals have continued to prevent the leakage of free water to the subgrade despite the wide separations that followed the steel failures.

Of particular interest is the complete absence of pumping at all joints in the halves of the two sections which contain shear bars and the 91-pound wire fabric. Considering that the special sections were constructed on a subgrade soil of proved susceptibility to pumping, it is indicated that structurally sound pavements of this design are highly resistant to pumping.

As would be expected, tilting of the 10-foot slabs and consequent faulting at the joints was quite evident at all weakened-plane joints at which the reinforcing steel failed. Because of pumping, this condition was most severe at those of the surface-groove type. Faulting in amounts up to ⅝ inch was measured at the impaired joints and, in several areas, the pavement had been patched with asphalt to restore surface smoothness.

Riding quality

During the last survey, motoring over the heavily traveled lane of the four special sections left the impression that the halves containing shear bars were of acceptable surface smoothness, but not as pleasantly smooth as the long stretches of pavement between joints of the heavily reinforced regular sections. On the other hand, the halves without shear units were, in places, disagreeably rough. The two halves reinforced with the 45-pound wire fabric, in which the majority of the steel failures occurred, were exceptionally bumpy.

Table 7.—Comparison of the roughness indexes of the various halves of the special sections with the average index of the pavement between joints of the long, regular sections [1]

Weight of reinforcement *Lb. per 100 sq. ft.*	Shear bars	Ratio of roughness index of a given half of a special section [2] to that of pavement between joints of long, regular sections	
		Outer lane	Inner lane
SURFACE-TYPE JOINT			
91	Yes	1.2:1	1.2:1
91	No	1.8:1	1.3:1
45	Yes	1.2:1	1.2:1
45	No	3.2:1	1.6:1
SUBMERGED-TYPE JOINT			
91	Yes	1.3:1	1.3:1
91	No	1.6:1	1.5:1
45	Yes	1.2:1	1.3:1
45	No	2.1:1	1.6:1

[1] All roughness indexes were determined at pavement age of 15½ years.
[2] Each roughness index is based on 250 feet of pavement.

Roughness indexes of the four 500-foot special sections were determined at the same time as those of the regular sections. These indexes provide a means for evaluating the effects on surface roughness of the features listed in table 5. Comparisons of the roughness index of each half of each of the four special sections with the average index established for the pavement between joints of the long, heavily reinforced regular sections are given in table 7. All roughness indexes compared in this table were determined at pavement age of 15½ years.

Considering the heavily traveled outer lane, it is apparent from table 7 that the surfaces of the four halves of the special sections containing shear bars in the weakened-plane joints were, after 15½ years: (1) 20 to 30 percent rougher than the surface of the pavement between the joints of the long, regular sections, (2) of nearly the same roughness, irrespective of type of joint or the amount of longitudinal steel crossing the joints, and (3) smoother, markedly so in two cases, than the surfaces of the four halves with no shear units in the joints.

A comparison of surface roughness confined specifically to the heavily traveled lane and the four halves of the special sections without shear bars in the weakened-plane joints indicates that, for comparable percentages of reinforcing steel, the halves containing the submerged type joint have, with time, less surface roughness than those containing the surface-groove type, and, for the same type of joint, the halves of sections reinforced with the 91-pound fabric have, with time, considerably smoother surfaces than those reinforced with the 45-pound fabric.

Interlane comparisons at the end of 15½ years disclose that traffic has had little effect on the surface roughness of the four halves of the special sections with shear bars in the joints. However, in the halves without shear units, the surfaces of the sections in the heavily traveled lane have, with one exception, become appreciably rougher with time than those of their companion sections in the passing or inner lane. The one exception is the half of the section containing submerged type joints and the 91-pound wire fabric. In the heavily traveled lane of this half only two steel failures occurred and pumping was completely absent.

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Revisions to the Manual on Uniform Traffic Control Devices

Important changes in the design of traffic signs and other traffic control devices have recently been published in *Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways*, printed for the Bureau of Public Roads by the Government Printing Office.

For many years the *Manual on Uniform Traffic Control Devices* has been the standard guide for American traffic engineers in the design and application of traffic control devices but, like any other standard, it must change to keep pace with a changing world. The responsibility for its periodic revision lies with the National Joint Committee on Uniform Traffic Control Devices, a group of experts representing the American Association of State Highway Officials, the Institute of Traffic Engineers, and the National Committee on Uniform Traffic Laws and Ordinances. The latest edition of the Manual was published in 1948. The Joint Committee met during 1953 to review the standards and to recommend revisions on the basis of another 5 years' experience. Its recommendations for revision have been approved by the three parent organizations and are now published in pamphlet form.

The *Manual on Uniform Traffic Control Devices for Streets and Highways, including the revisions supplement*, is sold by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 90 cents a copy. As a separate publication, the *Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways* is available from the same source at 15 cents a copy.

The principal changes brought about by the revisions, most of them affecting traffic signs, are as follows:

1. A red stop sign, with white lettering, and with at least the lettering reflectorized, replaces the former standard yellow sign. No contrasting panels or supplementary messages are permitted.

2. A "Yield Right of Way" sign is adopted for use at certain intersections where a full stop is not required. This sign is a yellow equilateral triangle, with one point downward.

3. "No Passing" and "End No Passing Zone" signs are replaced by signs reading "Do Not Pass" and "Pass With Care", respectively.

4. The cardinal direction marker is to read "North", "South", etc., rather than "Northbound", etc.

5. The minimum mounting height to the bottom of signs in rural areas is increased to 5 feet, to avoid road splatter.

6. Warning signs are to be placed further in advance of dangers ahead.

7. The border around signs is now optional rather than mandatory.

8. Railroad-crossing pavement markings are confined to the right-hand portion of the pavement.

9. Traffic signal installations must show at least two signal faces visible on each approach to an intersection.

10. The minimum traffic "warrants" (vehicular and pedestrian volumes) justifying traffic signal installation are raised in certain respects.

Construction Review

The Departments of Commerce and Labor have joined efforts in production of a new periodical, *Construction Review*, which will serve as a comprehensive and authoritative source of information on the construction industry. The new monthly publication, which will start with the January 1955 issue, replaces the Labor Department's *Construction and Building Materials*.

Construction Review is available by purchase from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. The subscription rate is $3.00 per year, and single copies are priced at 30 cents.

Construction Review brings together virtually all current statistics pertaining to construction which are compiled by the Federal Government, plus some nongovernmental material. Statistical series include value of new construction put in place, new nonfarm dwelling units started, building permit valuation, contract awards, construction cost indexes, wage scales, wholesale prices and production of building materials, contract construction employment, etc. The publication will also report on specific aspects of construction.

Annual Report of the Bureau of Public Roads, Fiscal Year 1954

The *Annual Report of the Bureau of Public Roads, Fiscal Year 1954* is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 30 cents a copy.

Covering the fiscal year ended June 30, 1954, the report deals with many vital aspects of highway transportation. As the principal roadbuilding agency of the Federal Government, the Bureau supervises the expenditure of funds authorized to aid the States in highway construction. It also supervises highway construction in National forests and parks and furnishes highway engineering assistance to other Federal agencies. In addition, the Bureau carries on a broad program of research dealing with the problems of highway improvement and traffic control.

The report discusses all phases of the Federal-aid construction program, which reached new high levels in 1954. Included are improvements on the National System of Interstate Highways, on primary and urban highways, and on farm-to-market roads. Outstanding Federal-aid projects are described. The report also reviews factors affecting progress, new highway legislation, foreign activities, including progress on the Inter-American Highway, and work in the field of highway safety.

The work done in fiscal year 1954 was financed by Federal-aid funds of $575 million authorized for the year and remaining balances of prior authorizations, together with State and local matching funds. All classes of Federal and Federal-aid projects completed during the year had a combined length of 20,989 miles. The decrease of 11 percent in total mileage completed from the all-time record of the previous year was due to a greater dollar volume of work in urban areas where costs per mile are high. The total cost of projects completed during the year amounted to $1,106 million, of which $585 million was Federal aid. The total was 2 percent greater than the previous year.

The 20,989-mile total included 4,488 miles of highways and 995 bridges on the Federal-aid primary system outside of cities (principal intercity routes), 767 miles of highways and 453 bridges on urban portions of the primary system, 14,995 miles of highways and 1,631 bridges on secondary or farm-to-market roads, and 739 miles of highways in National forests, parks, parkways, and flood-relie projects. At railway-highway grade crossing 193 hazardous crossings were eliminated, 2 inadequate grade-crossing structures were re constructed, and 315 crossings were protecte by the installation of flashing lights or othe safety devices.

Research activities covered by the repor include highway design standards, bridg design, and joint planning of location of high ways and airports, and development of high way improvement programs for urban areas A section of the report deals with parkin meters—their financing, revenues, installatio costs, and related matters. The report pre sents the results of physical research involvin soils and related problems directly affectin improved road construction and lower costs

In reviewing recent developments, the re port points out that the most significant tren of the year was the increasingly strong an widespread demand that steps be taken t make our highway systems adequate withi the shortest possible period—with particula emphasis on the need for improved mai arteries between urban areas and for express ways in cities.

Criteria for Prestressed Concrete Bridges

The Bureau of Public Roads recently issued a new publication, *Criteria for Prestressed Concrete Bridges*, which is available by purchase from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents a copy.

The criteria presented in the 31-page pamphlet were developed in the hope that they may be useful until such time as more complete specifications, covering the subject in far greater detail, may be presented to the civil engineering profession by American specification and code writing bodies.

Included in the publication are criteria on the design, materials, and construction of prestressed concrete bridges, and explanatory discussions of the criteria.

The Bureau recognized in 1952 that the prestressed method of concrete construction had great possibilities in the building of better and more economical highway bridges of reinforced concrete, and that in many instances prestressed concrete might become a competitor of structural steel also.

There were no American standard codes governing the design of prestressed concrete bridges at that time. In recognition of the need for a guide to design that would provide structures acceptable for Federal-aid projects, the Bureau, in March 1952, prepared and distributed a *Design Criteria for Prestressed Concrete Bridges (Post Tensioning)*. Although the scope of the criteria were very limited the issue attracted considerable attention and many constructive comments and sug gestions were received from American an European engineers engaged in prestresse concrete design and construction.

On the basis of these comments and sug gestions, a rough draft of a new and greatl enlarged criteria, covering design, material and construction, was prepared and sub mitted in September 1953 to a number o authorities in the field both in this countr and abroad. Thoroughly revised in the ligh of their comments, the criteria, together wit supporting discussion and source reference now appear in the new publication.

Uniform Vehicle Code

A new one-volume edition of the *Uniform Vehicle Code* was recently published by the National Committee on Uniform Traffic Laws and Ordinances. The Code, formulated in 1926 and revised periodically in the light of new developments and practical experience, provides a reliable guide for the use of the legislatures of the States in the development of Nationwide uniform traffic regulation.

The *Uniform Vehicle Code* (revised 1954) is a consolidation, rearrangement, and revision of the material heretofore included in five separate acts, formerly published by the U. S. Government Printing Office. The Code is now incorporated in a single act of 19 chapters, and the new arrangement will facilitate study of present laws in comparison with the provisions recommended as a guide for uniformity. The table of contents provides a key to the section numbering in the previous five individual acts.

The new 197-page *Uniform Vehicle Code* is published and sold by the National Committee on Uniform Traffic Laws and Ordinances, 1604 K Street, N. W., Washington 6, D. C. Singl copies are priced at 75 cents, plus 8 cents fo postage and handling, making a total charg per copy of 83 cents. The Committee, o request, will quote prices for quantity orders

The *Model Traffic Ordinance* (revised 1953) companion publication to the Uniform Vehicl Code with particular application to cities continues to be available from the Superin tendent of Documents, U. S. Governmen Printing Office, Washington 25, D. C., at 2 cents a copy.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

FOR OFFICIAL USE

AS OF DECEMBER 31, 1954

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$16,148	$15,533	$8,172	461.6	$4,223	$2,539	20.2	$43,967	$21,942	544.2	$63,723	$32,653	1,026.0
Arizona	5,998	8,535	5,747	126.2	1,124	764	4.1	6,611	4,610	128.0	16,270	11,141	258.3
Arkansas	13,076	8,045	4,539	325.4	6,047	3,172	173.6	14,336	7,078	300.6	28,428	14,789	799.6
California	14,012	30,646	16,180	167.6	15,536	8,216	55.9	118,824	58,204	294.5	165,006	82,600	518.0
Colorado	15,244	6,999	4,046	112.7	3,203	1,748	60.9	15,307	8,359	135.0	25,509	14,153	308.6
Connecticut	16,218	537	268	1.7	2,861	1,414	3.2	9,927	4,769	8.2	13,325	6,451	13.1
Delaware	4,658	841	422	4.0	2,582	1,285	9.5	6,597	3,656	23.3	10,020	5,363	36.8
Florida	13,280	19,016	9,737	222.4	7,466	3,853	50.2	20,541	10,558	337.6	47,025	24,148	610.2
Georgia	19,622	12,118	6,153	220.0	16,172	6,849	73.1	48,661	23,417	929.7	76,951	36,419	1,222.8
Idaho	4,292	9,657	6,163	130.4	2,489	1,549	50.5	12,125	7,633	170.7	24,271	15,345	351.6
Illinois	36,349	39,876	20,995	540.1	11,213	5,616	59.0	63,993	33,592	306.7	115,082	60,203	905.8
Indiana	20,917	38,462	19,263	168.4	18,661	9,566	128.1	25,618	13,809	92.4	82,741	42,638	388.9
Iowa	12,431	17,713	9,557	444.4	8,398	4,520	146.1	16,967	9,401	757.5	43,078	23,478	1,348.0
Kansas	16,160	9,384	4,759	744.0	3,293	1,671	60.0	21,596	11,102	962.2	34,273	17,532	1,766.2
Kentucky	15,080	9,988	5,406	61.4	4,769	2,433	102.8	25,481	12,807	225.1	40,258	20,646	389.3
Louisiana	15,803	10,409	5,193	135.4	8,860	4,430	33.8	23,636	10,656	102.8	42,905	20,279	272.0
Maine	6,242	7,102	3,801	41.3	1,655	834	20.0	14,453	7,309	115.1	23,210	11,944	176.4
Maryland	11,483	12,876	6,567	53.6	7,938	3,830	15.6	8,454	4,685	51.3	29,268	15,082	120.5
Massachusetts	16,071	3,754	1,867	20.2	8,146	4,005	6.7	52,575	24,689	42.4	64,475	30,561	69.3
Michigan	23,633	30,350	18,946	468.9	9,274	4,665	129.4	34,980	17,667	207.6	80,604	41,278	805.9
Minnesota	22,084	9,533	4,990	552.7	3,164	1,754	97.0	15,876	8,565	390.1	28,573	15,309	1,039.8
Mississippi	9,544	15,037	7,337	497.1	5,691	2,976	154.0	20,745	10,663	405.4	41,473	20,976	1,056.5
Missouri	15,844	29,837	15,742	1,094.1	6,582	3,728	30.7	57,245	29,260	747.7	93,664	48,730	1,872.5
Montana	15,657	11,838	7,163	270.3	3,202	1,961	47.2	22,092	13,585	425.2	37,132	22,709	742.7
Nebraska	18,182	16,360	8,579	720.6	9,725	5,720	421.9	15,538	8,542	525.4	41,623	22,841	1,367.9
Nevada	10,535	5,610	4,744	130.4	719	599	15.0	3,757	3,122	41.5	10,086	8,465	186.9
New Hampshire	5,851	1,958	1,154	10.5	284	145	1.6	6,421	3,137	35.3	8,663	4,436	47.4
New Jersey	22,614	8,025	4,012	43.8	3,233	1,999	3.4	15,380	7,096	29.1	26,638	12,703	76.3
New Mexico	7,035	6,234	3,845	198.4	2,225	1,434	15.0	9,613	6,136	159.5	18,072	11,415	372.9
New York	45,080	47,215	24,766	84.7	49,686	24,955	18.1	200,108	93,170	338.0	297,009	142,891	440.8
North Carolina	21,675	10,519	7,842	315.6	4,010	1,939	63.5	39,899	18,596	475.5	60,428	28,377	854.6
North Dakota	16,382	6,869	4,520	842.0	877	452	144.2	4,906	2,462	281.6	14,652	7,434	1,267.8
Ohio	31,[illegible]	30,[illegible]	14,849	74.8	10,018	4,075	18.3	72,242	34,439	158.8	112,526	53,363	251.9
Oklahoma	22,869	7,305	4,128	180.0	10,067	5,273	133.0	19,638	10,264	312.2	37,010	19,665	625.2
Oregon	[illegible],970	4,665	2,763	61.1	3,912	2,262	51.8	11,194	6,830	132.7	19,411	11,861	245.6
Pennsylvania	47,122	1,613	807	1.6	22,043	11,193	20.8	83,967	41,498	159.7	108,423	53,498	182.1
Rhode Island	3,820	[illegible],935	4,467	32.0	2,527	1,263	1.5	4,653	2,434	16.8	10,317	8,164	50.3
South Carolina	12,515	10,757	5,784	187.0	3,589	1,779	20.1	14,741	7,727	259.7	29,094	15,290	466.8
South Dakota	7,428	10,169	5,151	703.8	2,096	1,174	132.7	6,134	3,470	290.6	24,399	13,795	1,127.1
Tennessee	20,439	12,907	6,407	300.7	4,613	2,306	54.8	33,067	15,020	329.6	50,587	23,733	685.1
Texas	43,678	10,800	6,368	247.2	9,441	4,922	130.0	72,583	38,568	1,111.0	92,824	49,858	1,488.2
Utah	8,720	2,800	2,080	46.9	2,556	1,907	62.3	3,616	2,837	21.0	8,972	6,824	130.2
Vermont	4,849	1,134	570	17.6	429	224	2.2	7,113	3,542	62.0	8,676	4,336	81.8
Virginia	17,275	11,317	5,700	179.4	6,844	3,399	117.7	21,315	9,906	138.4	39,476	19,005	435.5
Washington	13,711	8,650	4,994	80.1	2,681	1,425	59.5	17,268	8,873	113.1	28,599	15,292	252.7
West Virginia	12,434	11,313	5,745	78.8	2,545	1,287	3.4	13,614	6,824	51.6	27,472	13,856	133.8
Wisconsin	18,866	21,265	10,735	260.1	521	300		24,907	12,587	262.5	46,693	23,622	522.6
Wyoming	5,601	5,109	3,287	99.1	1,738	1,151	15.6	7,507	4,708	146.0	14,374	9,146	260.7
Hawaii	4,519	2,980	1,490	2.8	1,058	351	2.2	5,788	2,715	15.1	9,826	4,556	20.1
District of Columbia	5,642	5,934	2,967	7.9	2,132	1,055	.6	13,620	6,477	1.2	21,686	10,499	9.7
Puerto Rico	9,663	6,557	3,193	39.2	1,187	572	4.2	15,382	6,867	42.4	23,126	10,632	85.8
TOTAL	807,714	655,294	347,966	11,810.0	323,845	166,155	2,745.0	1,444,787	725,863	13,213.6	2,423,926	1,239,984	27,768.6

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APRIL 1955

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Examining portland cement concrete slabs for scaling action

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The printing of this publication has been approved by the Director of the Bureau of the Budget, January 5, 1952.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

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Factors Affecting Resistance of Portland Cement Concrete to Scaling Action of Thawing Agents

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by ALBERT G. TIMMS, Supervising Highway Physical Research Engineer

This report is a resume to date of investigations conducted to test materials and procedures for protecting concrete pavements against scaling and disintegration caused by calcium chloride and other thawing agents used for ice removal. A laboratory investigation by the Bureau of Public Roads was started in 1948 on methods of protecting the wearing surface of concrete against the action of calcium chloride. Later a similar study was made of the effect of outdoor weather conditions on small slabs placed on the ground. It was found that resistance to scaling was affected by air content, type of air-entraining admixture, surface treatments or coats, admixtures of oils, inhibitors, fly ash as a replacement for portland cement, rate of application of calcium chloride, curing methods, thawing agents other than calcium chloride or common salt, and by the vacuum method of placing concrete.

DURING the winter months when ice has formed on pavements and bridge decks, sodium chloride or calcium chloride or mixtures of the two are spread on surfaces to thaw the ice and make the riding surface safe for traffic. This practice has been common in many parts of the country for 25 years or more. The continued use of these salts, usually spread in flake, pellet, or crystal form, has frequently resulted in excessive scaling of the wearing surface of portland cement concrete. In some cases the action has been so severe as to cause complete disintegration.

In States where ice covers the pavements many times each winter, the problem of ice removal by chemicals and the attendant scaling and disintegration of the concrete is a serious one. Since sodium and calcium chlorides are very effective for melting ice, there is great interest in developing methods of making concrete resistant to their action. In New York, particularly, and in several of the other Northeastern States, climatic conditions, hilly terrain, and heavy traffic have intensified this problem of developing a concrete resistant to the scaling caused by chloride salts.

The Bureau of Public Roads began a laboratory investigation in 1948 of methods of protecting the wearing surface of concrete against the action of calcium chloride. Later because of the extreme severity of the laboratory test which caused doubt as to its similarity to field conditions, a similar study was made under outdoor weather conditions on small concrete slabs on the ground. This is a progress report of a continuing research program and has been prepared to summarize the information obtained on the laboratory tests and the outdoor exposure tests through the winters of 1951–52, 1952–53, and 1953–54.

Scope of Study

The laboratory investigation of scaling resistance was divided into five parts as follows:

1. The effect of increasing the air content of the fresh concrete beyond the 6 percent maximum limit now generally permitted in specifications for concrete for pavement construction was investigated.

2. The effect of delaying the scaling action by surface coatings of crankcase oil (undiluted and diluted) with various percentages of gasoline, and the effect of time of application of the oils were studied.

3. The effect on scaling action of using paraffin and asphaltic base lubricating oils (both new and used) as admixtures in concrete was studied, and also the effect of these materials on strength, shrinkage, and resistance to freezing and thawing in water.

4. An investigation was made of the following miscellaneous factors: Effect of using various amounts of calcium chloride applied to a given area for ice removal, of using a possible rust inhibitor mixed with calcium chloride, and of using urea for ice removal in place of calcium chloride.

5. A study of the improvement in the quality of the wearing surface produced by vacuum treatment of the concrete was undertaken.

The outdoor investigation to date covers tests over three winters. The variable studied during the first winter was type of air-entraining admixture. Twenty-seven commercial air-entraining admixtures were used in amounts that produced about the same air content in each of the concrete slabs. During the second and third winters the outdoor program was divided into four parts:

1. Tests made the first winter were repeated with most of the 27 air-entraining admixtures, but using two cements (brand A with high alkali content and brand B with low alkali content). Five more air-entraining admixtures were included which had been received too late for the first series. These concrete slabs were cast in molds both on a metal base and on a sand base.

2. The effect on the resistance to scaling of replacing part of the portland cement with fly ash was studied. A fine and coarse fly ash from each of two sources were used because previous experience indicated that the finer the fly ash from a given source the lower the carbon content. Two concretes, plain and air-entrained were used for each fly ash replacement, with each of the two different cements (brands A and B).

3. The effect of methods of curing the concrete on resistance to chloride attack was investigated. In connection with the curing study, surface applications of oil were also made following the curing period.

4. The effect of vacuum treating the surfaces of both plain and air-entrained concrete was studied.

Conclusions

The principal conclusions of the laboratory tests (artificial freezing) were as follows:

1. A scaling test which involved freezing of water on the surface of concrete test slabs and thawing with an application of flake calcium chloride showed that the resistance of the concrete was a function of the amount of entrained air. An air content in excess of 6 percent was more effective in making concrete resistant to the scaling action of calcium chloride than increasing the cement content.

2. Concrete which was cured and seasoned and then coated with mineral oil showed greater resistance to calcium chloride attack than similarly cured concrete which received no protective treatment. Multiple coats of oil were slightly more beneficial than a single coat of oil. Application of oil to freshly placed concrete decreased the resistance to scaling.

229486 55

Table 1.—Chemical composition and physical properties of portland cements and fly ashes

	Portland cement		Fly ash			
	A	B	A	B	X	Y
Chemical composition (percent):						
Silicon dioxide	22.0	22.3	47.1	49.2	41.2	38.5
Aluminum oxide	5.6	5.4	18.2	19.9	22.1	23.5
Iron oxide	2.5	2.4	19.2	16.2	20.6	18.8
Calcium oxide	62.8	66.1	7.0	5.5	6.0	3.2
Magnesium oxide	3.0	1.0	1.1	1.4	1.2	1.0
Sulfur trioxide	2.0	1.7	2.8	2.7	.9	.6
Loss on ignition	.8	1.2	[1] 1.2	[1] 1.2	[1] 5.4	[1] 11.6
Sodium oxide	.40	.04	1.80	2.00	1.00	.60
Potassium oxide	1.05	.15	2.15	2.35	1.42	1.88
Total equivalent alkalies as Na_2O	1.09	.14	3.21	3.55	1.93	1.84
Insoluble residue	.16	.12	----	----	----	----
Chloroform-soluble organic substances	.009	.003	----	----	----	----
Free calcium oxide	.85	.56	----	----	----	----
Water-soluble alkali:						
Na_2O	.11	.01	----	----	----	----
K_2O	.63	.02	----	----	----	----
Computed compound composition (in percent):						
Tricalcium silicate	42	55	----	----	----	----
Dicalcium silicate	31	22	----	----	----	----
Tricalcium aluminate	11	10	----	----	----	----
Tetracalcium aluminoferrite	8	7	----	----	----	----
Calcium sulfate	3.4	2.9	----	----	----	----
Carbon	----	----	.2	.6	5.0	11.2
Carbon dioxide	----	----	.01	.04	.04	.03
Physical properties:						
Apparent specific gravity	3.20	3.17	2.49	2.52	2.51	2.43
Specific surface (Wagner), cm.²/g.	1,800	1,625	----	----	----	----
Specific surface (Blaine), cm.²/g.	----	----	3,075	4,305	2,565	3,220
Autoclave expansion, percent	.32	.04	----	----	----	----
Normal consistency, do	25.5	25.0	----	----	----	----
Time of setting (Gillmore test):						
Initial, hours	3.2	4.2	----	----	----	----
Final, do	5.2	6.4	----	----	----	----
Compressive strength (1:2.75 mortar):						
At 7 days, p.s.i	2,340	2,960	----	----	----	----
At 28 days, do	3,670	5,070	----	----	----	----
Mortar air content, percent	9.1	6.8	----	----	----	----

[1] Determination made at 600° C.

3. Neither paraffin nor asphaltic base oil used as admixtures were of much value in either delaying or controlling the progress of scaling. Used crankcase lubricating oil was effective in retarding the start of scaling, because of the air entrained in the fresh concrete by this material.

The use of one-third or two-thirds of a gallon of paraffin or asphaltic base oil per sack of cement as an admixture had no effect on the air entrained in the concrete. A slight reduction in shrinkage and in strength and a slight improvement in durability as measured by resistance to freezing and thawing in water resulted from the use of these oils.

The use of one-third or two-thirds of a gallon of used crankcase lubricating oil per sack of cement entrained air in the concrete. Concretes containing these admixtures showed a reduction in strength and an improvement in durability proportional to the amount of air entrained. The shrinkage of the concrete containing crankcase oil was about the same as the concrete without admixture.

4. Urea, reported to be a thawing agent non-corrosive to metals when used to thaw ice on concrete, was slower in thawing action than calcium chloride. It also caused scaling but not so quickly as calcium chloride.

Varying the amount of the thawing agent (calcium chloride or urea) had little effect either on the start or rate of progress of the scaling. The use of a metal corrosion inhibitor had only a slight retarding action on scaling of concrete when used with calcium chloride for ice removal.

5. Concretes containing 6 or 7 sacks of cement per cubic yard, both plain and air-entrained, were benefited in their resistance to scaling by the use of the vacuum treatment of the plastic concrete.

The principal conclusions of the outdoor exposure tests were as follows:

1. All the air-entraining admixtures tested in concrete were effective in delaying the start of serious scaling. As indicated by the preliminary tests of the first winter, the synthetic detergents and the salts of proteinaceous materials were less effective than the admixtures in the other groups. In the subsequent tests made the next two winters, the synthetic detergents were relatively more effective than they were in the preliminary tests. For the concrete containing aggregates with a 1-inch maximum size, the tests indicate that more satisfactory resistance is obtained when the air content is greater than 5 percent.

2. For the variables studied, the scaling of concrete was less pronounced when the concrete had been cast in a mold with a sand base than when cast in a mold with a metal base. The water retention of the concrete cast on the sand base is less than that cast on a metal base. The resulting decrease in the water-cement ratio accounts for the difference in resistance to scaling.

3. The two portland cements used did not produce concretes of equal resistance. The concretes made with cement B (low alkali content) were more resistant than the concretes made with cement A (relatively high alkali content). The different treatments of the concretes did not appear to change this relative difference between the cements in resistance to scaling.

4. Replacing portland cement in the mix with fly ash, regardless of the fineness or carbon content, was detrimental to the resistance of the concretes to attack by calcium chloride used for ice removal. Maintaining

Table 2.—Mix data for laboratory slabs, 6 by 12 by 2 inches thick [1]

Mix by dry weight	Admixture		Cement	Water	Slump	Air	Weight of plastic concrete
	Amount [2]	Type					
PART 1: EFFECT OF AIR CONTENT							
Pounds			*Sack/cu. yd.*	*Gal./sack*	*Inches*	*Percent*	*Lb./cu. ft.*
94-210-320	----	None	6.0	5.7	2.3	1.5	149.0
94-200-320	0.004	Vinsol resin	6.0	5.4	2.4	2.7	148.0
94-180-320	0.011	do	6.1	5.1	2.6	6.0	143.7
94-160-320	0.025	do	5.9	4.7	3.2	13.0	131.6
94-170-270	----	None	7.0	4.8	2.6	1.5	149.3
94-155-270	0.005	Vinsol resin	7.2	4.5	2.4	2.7	148.9
94-140-270	0.011	do	7.2	4.2	2.2	4.9	143.8
94-130-270	0.022	do	7.1	4.1	2.2	7.2	139.6
PART 2: EFFECT OF OIL SURFACE TREATMENT							
94-210-320	----	None	5.9	6.1	3.5	1.9	147.3
PART 3: EFFECT OF OIL ADMIXTURES							
94-210-320	----	None	5.9	6.1	3.0	2.0	146.6
94-170-320	⅔gal.	Used crankcase oil	6.0	5.4	4.8	5.7	140.0
94-200-320	⅔gal.	Paraffin oil	5.9	5.7	4.0	2.0	146.3
94-200-320	⅔gal.	Asphalt oil	5.9	5.7	3.5	2.4	145.3
PART 4: EFFECT OF THAWING AGENT							
94-210-320	----	None	5.9	6.1	3.5	1.9	147.3
PART 5: EFFECT OF VACUUM SURFACE TREATMENT							
94-210-320	----	None	6.0	5.7	----	1.7	148.6
94-160-320	0.020	Vinsol resin	6.2	4.7	----	7.4	141.1
94-170-270	----	None	7.0	4.8	----	1.7	148.6
94-130-270	0.022	Vinsol resin	7.1	4.1	----	7.8	139.3

[1] Materials used: cement, brand A; siliceous sand, F.M. =2.70; siliceous gravel, ¾-inch maximum size.
[2] Per sack of cement. Except for oil admixtures, amounts are expressed as percent by weight of cement.

Figure 1.—Rating scale (0–10) of resistance of concrete to scaling after laboratory freezing and thawing with calcium chloride.

fixed air entrainment within the usual pecification limits did not balance the lowered sistance caused by the use of fly ash.

5. In general, the type of curing had little pparent effect on the resistance of the con- retes to calcium chloride attack. The mem- rane curing, when the film remained un- roken, had some protective action. Under affic conditions such a film would probably e of little or no value.

6. The concrete cast in molds with metal ases and subjected to a vacuum treatment as little different in resistance to scaling om similarly cast untreated concrete. How- ver, when the concretes were cast in molds ith sand bases the concrete on which the acuum treatment was used was more resist- nt to scaling than the untreated concrete. his same relationship held for concrete with ir contents ranging from 1 to 10 percent.

Laboratory Exposure Series

The portland cement used was an ASTM ype I cement. It is designated as brand A ement and is the same cement as brand A sed in the outdoor investigation. Table 1 ives the chemical composition and physical roperties of the cement.

The aggregates used for all concrete mixes or the scaling tests consisted of a siliceous nd having a fineness modulus of 2.70 and a ell-graded siliceous gravel of ¾-inch max- num size. For the test specimens used for etermining strength, resistance to freezing nd thawing in water and volume change, rushed limestone was used as coarse aggre- ate. Concretes made from similar aggre- ates have good service records for durability. he concrete mix data are given in table 2.

The specimens used in the laboratory scaling est consisted of concrete slabs having a earing surface of 6 by 12 inches and a thick- ess of 2 inches. The slabs were cast and the oncrete rodded and spaded in the usual anner. In one group of slabs a vacuum reatment was applied to the plastic concrete fter casting.

Approximately 3 hours after molding, the op surface of each specimen was given a broomed finish to simulate the surface finish frequently given pavement slabs. Subsequent to brooming a mortar dam approximately one-half inch in height was cast around the perimeter of the specimen. In general, the concrete was cured in moist air, the bottoms and sides of the specimens were waterproofed with a heavy coating of paraffin, and then stored in the air of the laboratory before starting the freezing and thawing cycle. The exact period of curing is shown in the notes of tables 3–6 and 12–13.

The top surfaces of the slabs were first flooded with one-quarter inch of water, after which the specimens were placed in the freezer and the surrounding air temperature reduced to −10° F. The slabs were kept in the freezer approximately 15 hours, then removed from the freezer, and flake calcium chloride applied directly to the ice-covered surface. In general, the amount of calcium chloride applied was 2.4 pounds per square yard of ice-encrusted surface. This is the maximum amount usually applied in practice and is the amount used by other investigators. In one group of tests different amounts of calcium chloride were applied per square yard. After the ice had thawed, the calcium chloride solution was washed from the surface of the slabs, fresh water applied and the cycle repeated. One cycle was completed each 24 hours from Monday through Friday. The slabs remained in the freezer from Friday night until Monday morning.

In general, three slabs were made for each condition and two or more rounds of slabs made on different days for each condition of test. Only one round of slabs was photographed and since the rounds in general checked each other very closely the results of only one round are reported.

The various slabs were rated periodically for surface scale. The ratings were based on visual observation of the extent and depth of scale. The following tabulation describes the numerical significance of the rating:

0—No scale
1—Scattered spots of very light scale
2—Scattered spots of light scale
3—Light scale over about one-half of the surface
4—Light scale over most of surface
5—Light scale over most of surface, few moderately deep spots
6—Scattered spots of moderately deep scale
7—Moderately deep scale over one-half of the surface
8—Moderately deep scale over entire surface
9—Scattered spots of deep scale, otherwise moderate scale
10—Deep scale over entire surface

Typical examples of the various ratings are shown in figure 1, and figure 2 illustrates progressive scaling of three identical slabs as the number of cycles of freezing and removal of ice with calcium chloride was increased.

Outdoor Exposure Series

The portland cements used were ASTM Type I cements. Two cements were used, brand A and brand B. Brand A had a relatively high alkali content and brand B a very low alkali content. Table 1 gives the chemical and physical properties of the cements.

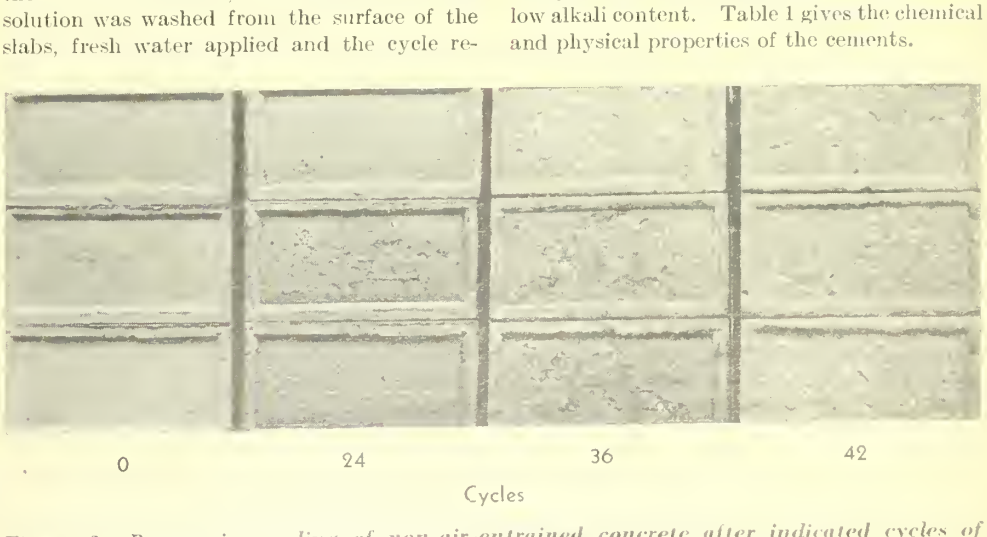

Figure 2.—Progressive scaling of non-air-entrained concrete after indicated cycles of laboratory freezing and thawing with calcium chloride.

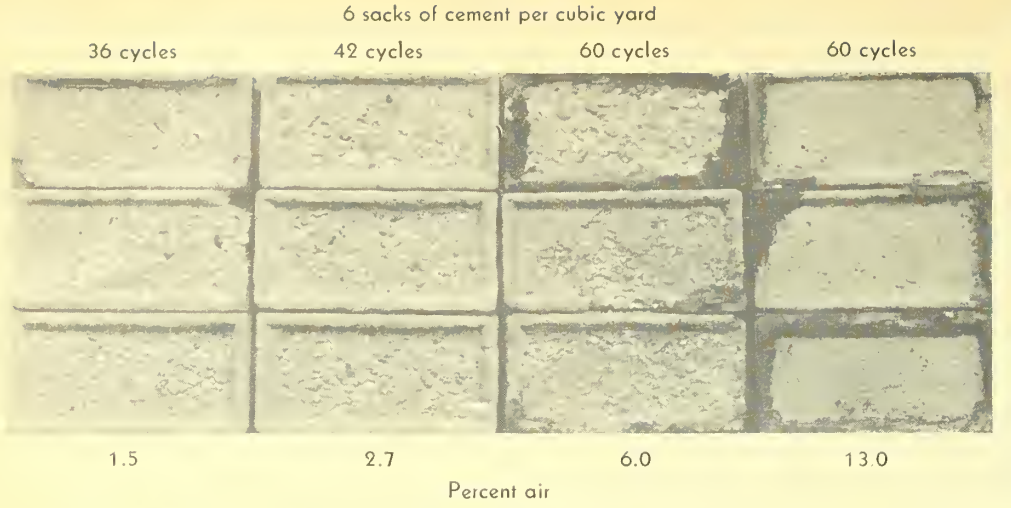

Figure 3.—Effect of percentage of entrained air on resistance of concrete to scaling after laboratory freezing and thawing with calcium chloride.

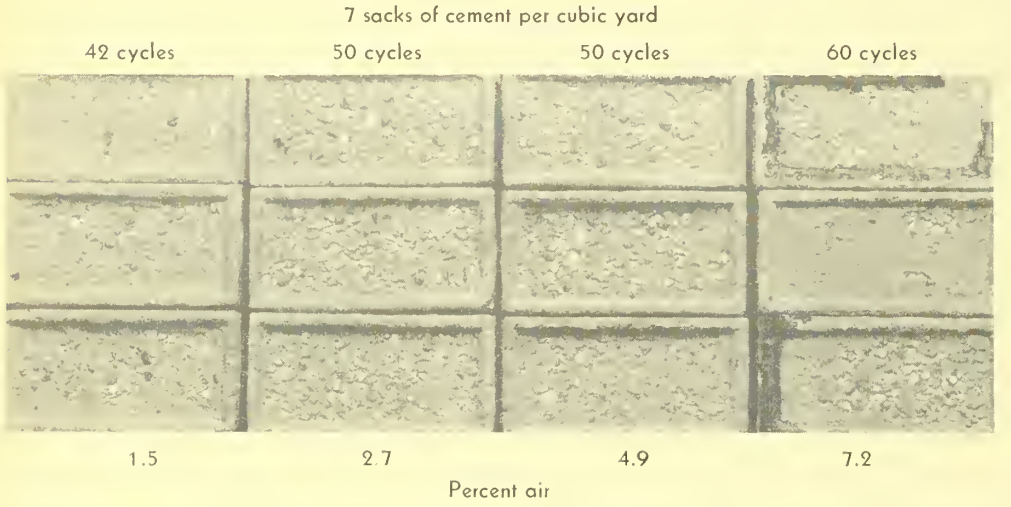

Figure 4.—Effect of percentage of entrained air on resistance of concrete to scaling after laboratory freezing and thawing with calcium chloride.

The aggregates used for all mixes consisted of a siliceous sand having a fineness modulus of 2.70 and a crushed limestone coarse aggregate of 1-inch maximum size. Concretes containing these aggregates have a good service record for durability.

The rapidity with which scaling started on the small slabs frozen in the laboratory freezer, suggested the possibility that the conditions were far more severe than those occurring under field applications of calcium chloride. In order to study this feature further, slabs 16 by 24 by 4 inches deep with raised edges or dams around the perimeters were made for outdoor exposure.

The surfaces of the slabs were given a broomed finish and most of them were then cured in the moist room from 28 to 90 days and then placed in the exposure area. The specimens were in the exposure area from 30 to 100 days before the first application of calcium chloride was made. A description of the treatment is in the notes to tables 15 to 21. Broomed finishing was selected because observations indicated that a broomed finish tended to hold the calcium chloride solution on the pavement surface and retarded its removal by drainage. In the fall and winter, the top surfaces of the test specimens were kept covered with water.

The first slabs were made in the laboratory in water-tight molds with a metal base. This condition simulates concrete placed on an impervious subgrade or on paper or asphalt seals placed on the subgrade. Later tests were made using a damp sand base which would be more nearly comparable to types of subbases often used under concrete pavements. With the exception of the series in which curing was studied, all other slabs were made in the laboratory.

When ice was frozen on the slabs, calcium chloride was applied to the surface at the rate of 2.4 pounds per square yard. After the ice was completely thawed, the surface was washed and fresh water left on the surface to await another freezing.

During the winter of 1951–52, 19 cycles were obtained, and during the winter of 1952–53, only 17 cycles, and in the winter of 1953–54, 34 cycles.

The slabs were rated periodically for surface scale. The ratings were based on visual observation of the extent and depth of scale. The numerical significance of the rating system used was the same as that shown for the small laboratory exposed slabs.

Discussion of Laboratory Tests

Air content, part 1

In this group of laboratory tests the object was to study the effect of increasing the air content beyond the maximum 6 percent limit now generally permitted for use in concrete pavements. Since the maximum size of aggregate used with the 6- by 12- by 2-inch slabs was three-fourths inch, this necessitated a higher air content for a given degree of durability than is required by a normal paving mix containing aggregate graded up to 1½ inches. This increase in air requirement has been shown by a number of investigators to be necessary in maintaining the level of durability.[1]

Two different proportions were used, one containing 6 and the other 7 sacks of cement per cubic yard. The slump of the concrete was maintained at 2 to 3 inches. Where entrained air was desired, neutralized Vinsol resin was used to produce the quantity of air specified.

Table 3 shows the scale ratings of the surfaces of the slabs containing various percentages of air. These ratings are reported at 30, 36, 42, 50, and 60 cycles when the tests were discontinued. Some tests were discontinued sooner because of the condition of the slab. The slabs having a cement content of 6 sacks per cubic yard had a maximum air content of 13 percent instead of the 7 to 8 percent which was planned.

The surface condition of the slabs after various cycles of exposure to calcium chloride action is well illustrated in the photographs in figures 3 and 4. Figure 3 shows the condition of the surface of concrete made with 6 sacks of cement per cubic yard and various air contents ranging from 1.5 to 13.0 percent. In each case the three slabs in a vertical column were identical in composition and

[1] *Effect of entrained air on concretes made with so-called "sand gravel" aggregates*, by Paul Klieger, Journal of the American Concrete Institute, Oct. 1948.

Table 3.—Rating of resistance to surface scaling of concrete slabs containing various percentages of air, part 1 of laboratory tests [1]

Cement	Air [2]	Rating after freezing and thawing [3] for—				
		30 cycles	36 cycles	42 cycles	50 cycles	60 cycles
Sack/cu. yd.	*Percent*					
6.0	1.5	8	10	---	---	---
6.0	2.7	6	8	10	---	---
6.1	6.0	2	3	5	6	8
5.9	13.0	0	0	0	2	2
7.0	1.5	3	7	10	---	---
7.2	2.7	2	5	8	10	---
7.2	4.9	0	2	5	10	---
7.1	7.2	0	1	2	4	6

[1] Each value is average of 3 tests. Slabs cured in moist air for 21 days followed by 14 days storage in laboratory air.
[2] Air content determined by ASTM tentative method C231–49T.
[3] Freezing and thawing tests were discontinued when surface scaling rating was 10, or at 60 cycles.

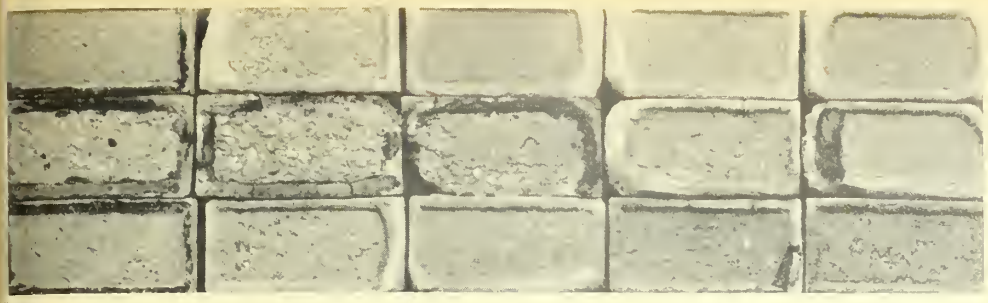

Figure 5.—Effect of single coat of mineral oil on resistance of non-air-entrained concrete to scaling after 40 cycles of laboratory freezing and thawing with calcium chloride.

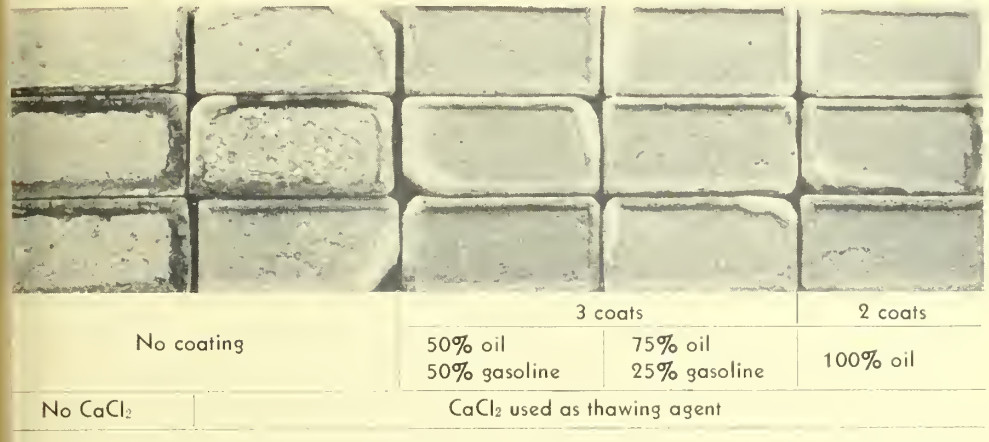

Figure 6.—Effect of multiple coats of mineral oil on resistance of non-air-entrained concrete to scaling after 40 cycles of laboratory freezing and thawing with calcium chloride.

eatment. Under each column is shown the r content of the fresh concrete.

The slabs in which no air-entraining admixture had been added (1.5 percent air) ere removed from test at 36 cycles because ıe surfaces were rated 10 after 36 cycles (see ble 3), and the entire slab was almost cometely disintegrated. Likewise the test of ıe concrete that contained only 2.7 percent r was discontinued after 42 cycles, at which ıne they were rated 10. It is interesting to ote that at 60 cycles 6 percent of entrained r, which is the maximum for most specifications, was not enough to give adequate protection (rating of 8) for this type of exposure. owever, 6 percent delayed the start of scaig and the rate of disintegration was less. he slabs shown in figure 3 containing 13.0 ercent were rated 2 after 60 cycles which is ttle more than the start of scaling. The igh air content was accidental as it was not ıe intention to exceed 8 percent. The test as made even though it was realized that ıe strength and wear resistance would be riously affected by the very high air content.

Figure 4 shows the slabs made with concete containing 7 sacks of cement per cubic ard. The range in air content was from 1.5 7.2 percent For a given air content the oncretes containing 7 sacks of cement per cubic yard were only slightly more resistant than the 6-sack concretes.

The conclusion that can be drawn from the results of these tests is the importance of using as high an air content as possible without jeopardizing unduly strength or wear resistance. The tests also show that the air content is far more important in its influence on resistance to scaling than the cement content.

Surface treatments, part 2

In this group a study was made of the effect on scaling of coatings of crankcase oil, undiluted and diluted, with various percentages of gasoline. The effect of time of application of the oils on start of scaling was also investigated.

Observations made in the field on actual pavements indicate that scaling was less likely to occur on those areas in the middle of the traffic lane containing noticeable oil stain than in the wheel tracks. It is believed that the oil drippings from cars and trucks fill the voids and reduce the absorption of the calcium chloride solution.

The concrete used in the tests was made with non-air-entrained cement and contained 6 sacks per cubic yard. The mix data are given in table 2.

The slabs in the first section of the group were given a single coating of unused lubricating or mineral oil (SAE No. 10 grade) either undiluted or diluted with gasoline. These surface treatments were quite similar to those used in New York State.[2] The combinations of oil and gasoline used for surface treatment and the scale ratings after 15 and 40 cycles of exposure are given in table 4. The condition of these slabs after 40 cycles of freezing and thawing is shown in figure 5.

In the second section of this group, specimens were given several coats of oil. Two coats of the undiluted oil were used and three coats of the diluted oils. The concrete would not absorb more than two coats of the undiluted oil within a reasonable period of time (24 hours) without leaving an appreciable residual film on the surface. The condition of these slabs is shown in figure 6.

The two columns on the left in figure 5 illustrate the condition of the slabs without surface treatment. Calcium chloride was not applied to the slabs shown in the first column from the left and the ice was thawed in laboratory air at about 75° F. These slabs on which no calcium chloride was used showed some action and were given a rating of 4. Calcium chloride was applied to all the other slabs including the three shown in the second column that were uncoated. The specimens were all subjected to 40 cycles before the tests were discontinued. The uncoated specimens to which calcium chloride was applied were seriously disintegrated at the end of 40 cycles and the surface scale rating was 8.

All of the single-coated specimens showed slight scale and one of each group of three identical specimens was badly disintegrated. There was no very marked difference in protection given by the undiluted oil as compared to the different dilutions as indicated by the ratings which varied from 2 to 4.

The surface ratings for the slabs given multiple coats of No. 10 oil or dilutions of the oil with gasoline are given in table 4, and the condition of the slabs after 40 cycles of freezing and thawing is shown in figure 6. The ratings indicate that at 40 cycles the multiple

[2] *Control of concrete pavement scaling caused by chloride salts*, by B. D. Tallamy, Journal of the American Concrete Institute, Mar. 1949, vol. 20, No. 7.

Table 4.—Effect of oil surface coatings on the resistance of concrete to scaling, part 2 of laboratory tests [1]

Number of applications	Surface treatment [2]	$CaCl_2$ for thawing	Rating after freezing and thawing for—	
			15 cycles	40 cycles
----	None	No	2	4
----	None	Yes	4	8
1	50% SAE 10 oil, 50% gasoline.	Yes	2	4
1	75% SAE 10 oil, 25% gasoline.	Yes	2	3
1	100% SAE 10 oil	Yes	2	2
----	None	No	2	4
----	None	Yes	4	8
3	50% SAE 10 oil, 50% gasoline.	Yes	2	2
3	75% SAE 10 oil, 25% gasoline.	Yes	2	3
2	100% SAE 10 oil	Yes	2	3

[1] Each value is average of 3 tests. Slabs cured in moist air for 21 days followed by 14 days storage in laboratory air.
[2] Surface treatment applied at the rate of 1 gallon per 20 square yards at age of 28 days.

oil treatment is slightly more beneficial in preventing scaling than the single coating.

Examination of all the slabs in the series indicates that no type of coating will prevent scaling from becoming progressive after a single break in the surface permits the calcium chloride solution to enter the concrete under the oil-impregnated layer. On pavements in service such breaks in the oil-protected surface may be caused by tire chains.

It was proposed in New York State that the oil coat could be applied to plastic concrete in lieu of a membrane curing compound. Therefore, to obtain information on the effect of such application, oil was applied to three slabs 3 hours after molding to simulate the time of application of a membrane curing compound. The other six slabs were given 7 days moist curing. Three of the six were given no further treatment. To the remaining three an oil surface coat was applied after 7 days drying. The ratings of these slabs after 15, 25, 50, and 60 cycles of freezing and thawing are shown in table 5. The application of oil to the surface of plastic concrete was definitely detrimental and resulted in much more severe scale than similar concrete that received no surface treatment. The slabs on which the oil was applied to the plastic concrete were rated 10 and the ones on which no oil was used were rated 6 after 60 cycles of freezing and thawing.

The concrete given an oil treatment after 14 days had far better resistance to scaling than that which received no earlier surface treatment. These slabs were rated 3. These tests indicate that the oil protective coat cannot be applied at an early age and still have value in improving resistance to scaling caused by application of calcium chloride.

Admixtures of oils, part 3

In this phase of the investigation, paraffin base oil, asphalt base oil, and used crankcase oil were used as admixtures in concrete. The mixes and the mix data for the concrete used in this group are given in table 2. The scale ratings of the slabs are shown in table 6 after 20, 30, 50, 65, and 75 cycles of freezing and thawing with calcium chloride. Figure 7 shows the specimens after 75 cycles of freezing and thawing.

The paraffin base oil and the asphalt base oil were ineffective in delaying the start of scaling or in controlling the rate of progress of the scaling. As may be seen from the table, the slabs containing these oils showed more scaling at 20 cycles than the slabs made without admixture. At 75 cycles the slabs containing the paraffin or asphaltic base oils were rated 10, the same as the concrete slabs without admixture.

The used crankcase oil was effective in retarding the start of scaling probably because of the air entrained in the plastic concrete. The concrete containing this material was rated 1 after 50 cycles and 4 after 75 cycles.

Since some of the materials used as admixtures are of value in delaying the start of scaling, the effect of these admixtures on other properties of concrete is important. To study these properties, concretes containing these admixtures were tested for flexural and compressive strength, durability as measured by freezing and thawing (specimens frozen in water), and volume change due to drying.

The mix data for the strength specimens are shown in table 7. Seven mixes were used, one a base mix without admixture containing 6.0 sacks of cement per cubic yard of concrete with a slump of approximately 3 inches. The other 6 mixes contained the admixtures and were similar to the base mix, except that the sand and water content was reduced to maintain approximately the same slump and cement content.

The one-third gallon of oil was selected because it was the amount used in previous tests for waterproofing concrete. Twice this amount was also used to determine if there were any harmful effects from using more than that recommended.

The air contents of the mix without admixture and of the mixes containing the paraffin and asphalt base oils ranged from 1.0 to 1.2 percent. The air content for the mix containing one-third gallon of used crankcase lubricating oil per sack of cement was 6.4 percent, and for the mix containing two-thirds of a gallon it was 5.0 percent. Air determinations were made using a pressure type air-meter.

The water required per sack of cement for the mixes containing the paraffin base and asphalt base oils was only slightly less than that required for the plain mix for the same slump and cement content. For the mix containing one-third gallon of used crankcase lubricating oil, it was 0.3 of a gallon less, and for two-thirds of a gallon it was 0.5 of a gallon less.

The workability of all of the mixes containing the admixtures was better than that of the plain concrete. This improvement was greater for those mixes which entrained air.

For each mix, eight 6- by 6- by 21-inch beams and eight 6- by 12-inch cylinders were made, two each on 4 different days. Four beams and four cylinders were tested at 7 days and four at 28 days. All specimens were stored continuously in moist air until tested.

Strength Tests

The results of the strength tests are given in table 8. The table also shows the ratio of the strength developed with admixture

Table 5.—Effect of time of application of oil coating on the resistance of concrete to scaling, part 2 of laboratory tests [1]

Surface treatment	Rating after freezing and thawing for—			
	15 cycles	25 cycles	50 cycles	60 cycles
None	3	4	5	6
Oil applied after 3 hours	4	4	6	10
Oil applied after 14 days [2]	2	2	3	3

[1] Each value is average of 9 tests.
[2] Slabs cured in moist air for 7 days followed by 28 days' storage in laboratory air followed by 14 days' soaking.

Table 6.—Rating of resistance to surface scaling of concrete slabs containing oil as admixture, part 3 of laboratory tests

Admixture	Air [2]	Rating after freezing and thawing for—				
		20 cycles	30 cycles	50 cycles	65 cycles	75 cycles
	Percent					
None	2.0	1	2	3	9	10
Used crankcase oil	5.7	1	1	1	2	4
Paraffin oil	2.0	3	4	4	10	10
Asphalt oil	2.4	3	5	5	10	10

[1] Each value is average of 5 tests. Slabs cured in moist air for 14 days followed by 40 days' storage in laboratory air.
[2] Air content determined by ASTM tentative method C231–49T.

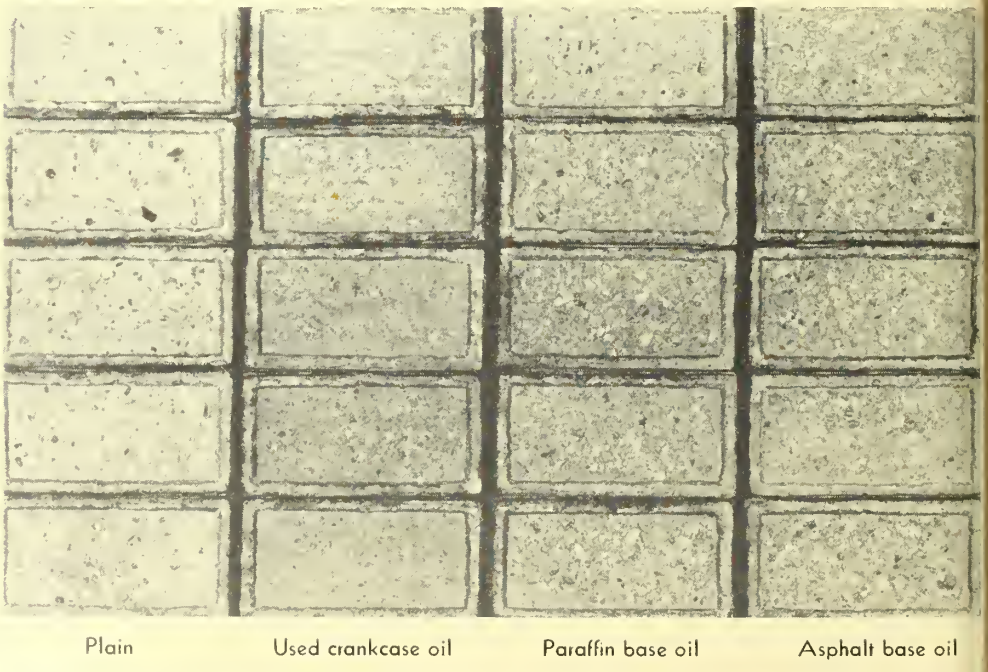

Figure 7.—Effect of oil admixtures on resistance of concrete to scaling after 75 cycles of laboratory freezing and thawing with calcium chloride.

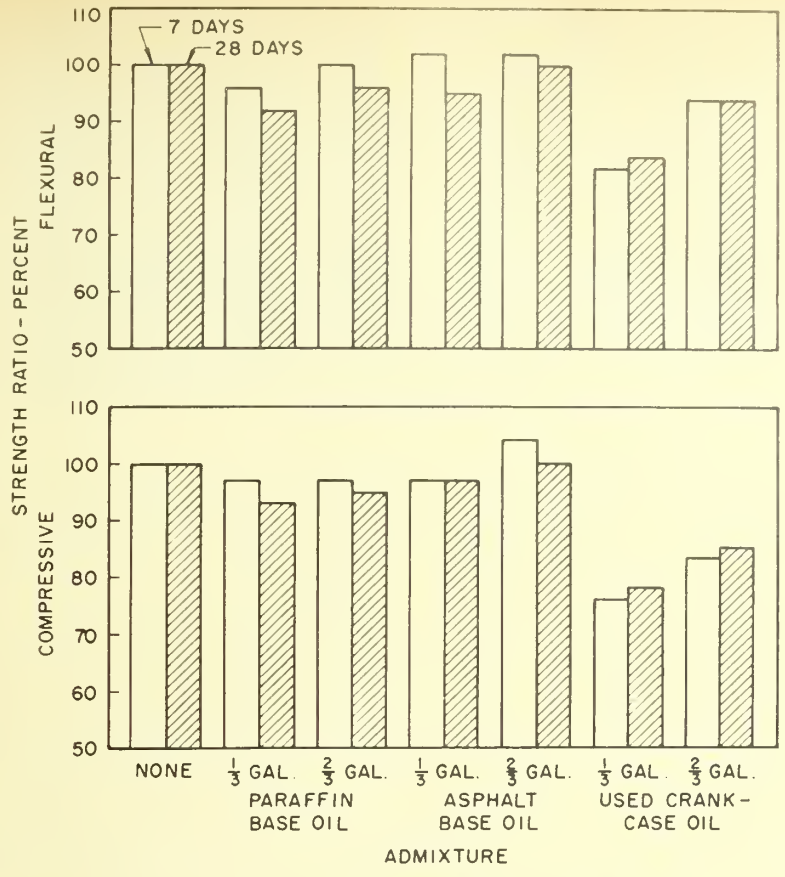

Figure 8.—Strength ratios of concrete containing oils.

expressed as percentages of the strengths of the corresponding concrete without admixture. The strength ratios are shown graphically in figure 8.

It should be noted that the use of the admixtures included in this series resulted in reductions in the 28-day flexural and compressive strengths. The 28-day ratios for the paraffin and asphalt base oils series ranged from 92 to 100 for flexural strength and 93 to 100 for compressive strength. These reductions would not be considered serious if the use of the admixture resulted in an improvement in the durability of the concrete.

There was a greater reduction in the strengths of the mixtures containing used crankcase oils. The use of these admixtures resulted in the entrainment of air in the concrete; therefore the strength ratio of 85 percent specified in the Tentative Specifications for Air-Entraining Admixtures for Concrete, ASTM Designation C260–52T will serve as a basis for comparison.

The strength ratios for the mixtures containing one-third gallon of used crankcase oil per sack of cement were lower than the ASTM Standard, and for those containing two-thirds of a gallon the ratios were all above 85 percent except the compressive strength ratio at 7 days which was 83 percent. The lower strengths of the ⅓-gallon mixtures were probably due to the air content of 6.4 percent as compared to 5.0 percent for the ⅔-gallon mixtures.

The mix data for the freezing and thawing specimens and the volume change specimens are given in table 9. Five mixes were used, one base mix without admixture, containing approximately 5 sacks of cement per cubic yard of concrete with a slump of 3.5 inches. The other four mixes contained the admixtures and were similar to the plain mix except that sand and water contents were reduced to maintain approximately the same consistency and cement content.

The mixes and materials used were similar to those used in the strength tests except that the maximum size of coarse aggregate was 1 inch instead of 1½ inches, and the percentage of sand was increased. The use of a smaller maximum size coarse aggregate resulted in higher air contents in the concretes than those in the concretes used for the strength specimens. The greatest difference was in the mixtures containing two-thirds of a gallon of used crankcase oil. Five percent air was obtained for the concrete (1½-inch maximum size) for the strength specimens and 10 percent for the concrete (1-inch maximum size) for the freezing and thawing specimens. A mixture containing one-eighth gallon of used crankcase oil was included in the freezing and thawing series. This mix had an air content of 5.2 percent which is within the accepted limits.

For each mix, nine 3- by 4- by 16-inch beams were made. Three beams were used for freezing and thawing, three were used for control and were tested for flexural strength along with the freezing and thawing specimens, and three were used for drying shrinkage tests.

Table 7.—Effect of oils on properties of concrete, mix data for strength specimens [1]

Mix by dry weight	Admixture: Amount	Admixture: Type	Cement	Water	Slump	Air [2]	Weight of plastic concrete
Pounds	*Gal./sack*		*Sack/cu. yd.*	*Gal./sack*	*Inches*	*Percent*	*Lb./cu. ft.*
94–195–350	------	None	6.0	5.6	2.7	1.1	153.2
94–195–350	⅓	Paraffin base oil	6.0	5.5	3.2	1.0	152.8
94–190–350	⅔	do	6.0	5.3	2.7	1.2	152.3
94–195–350	⅓	Asphalt base oil	6.0	5.5	2.7	1.0	152.6
94–190–350	⅔	do	6.0	5.3	2.8	1.0	152.1
94–180–350	⅓	Used crankcase oil	5.8	5.3	3.1	6.4	145.0
94–170–350	⅔	do	6.0	5.1	2.5	5.0	146.4

[1] Materials used: cement, brand A; siliceous sand, F. M.=2.70; crushed limestone coarse aggregate, 1½-inch maximum size.
[2] Air content determined by ASTM tentative method C231–49T.

Table 8.—Effect of oils on the strength of concrete [1]

Admixture: Amount	Admixture: Type	Air [2]	Modulus of rupture [3]: 7 days Strength	7 days Ratio	28 days Strength	28 days Ratio	Compressive strength [4]: 7 days Strength	7 days Ratio	28 days Strength	28 days Ratio
Gal./sack		*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*	*P.s.i.*	*Pct.*
------	None	1.1	605	100	765	100	3,560	100	4,990	100
⅓	Paraffin base oil	1.0	580	96	705	92	3,450	97	4,660	93
⅔	do	1.2	605	100	735	96	3,450	97	4,720	95
⅓	Asphalt base oil	1.0	615	102	730	95	3,440	97	4,840	97
⅔	do	1.0	615	102	765	100	3,690	104	4,990	100
⅓	Used crankcase oil	6.4	495	82	640	84	2,720	76	3,880	78
⅔	do	5.0	570	94	720	94	2,940	83	4,250	85

[1] Each value is the average of 4 tests.
[2] Air content determined by ASTM tentative method C231–49T.
[3] Specimens were 6- by 6- by 21-inch beams tested in accordance with ASTM standard method C78–49. Beams tested with side as molded in tension. Ratio values for relative strength are based on the strengths for the mix without admixture.
[4] Specimens were 6- by 12-inch cylinders tested in accordance with ASTM standard method C39–49. Ratio values for relative strength are based on the strengths for the mix without admixture.

Table 9.—Effect of oils on properties of concrete, mix data for freezing and thawing and drying shrinkage specimens [1]

Mix by dry weight		Cement	Water	Slump	Air [2]	Weight of plastic concrete
Pounds		*Sack/cu. yd.*	*Gal./sack*	*Inches*	*Percent*	*Lb./cu. ft.*
94-220-378	None	5.9	6.3	3.3	1.2	149.9
94-205-320	Paraffin base oil	6.0	5.8	3.9	1.3	148.7
94-205-320	Asphalt base oil	6.0	5.8	3.8	1.3	148.9
94-200-320	Used crankcase oil	5.9	5.6	4.1	5.2	144.5
94-190-320	do	5.6	5.6	4.2	10.0	137.0

[1] Materials used: cement, brand A; siliceous sand, F. M.=2.70; crushed limestone coarse aggregate, 1-inch maximum size.
[2] Air determined by ASTM tentative method C231-49T.

The results of the freezing and thawing tests are shown in table 10 and figure 9. The bars (3 by 4 by 16 inches) for the freezing and thawing tests were stored in moist air for 28 days prior to the start of the test. The freezing and thawing tests were made in a manner similar to that described in a previous article.[3]

The sonic modulus (N^2) was determined on the specimens prior to freezing, and then after regular intervals of freezing and thawing the percentage decrease in N^2 was determined.[4] When a group of specimens showed an average decrease in N^2 of 40 percent, they were considered disintegrated, and freezing and thawing was then discontinued and flexural strength tests were made. On the remaining specimens freezing and thawing was discontinued at 70 cycles and flexural strength tests were made.

In table 10 are given the losses in N^2, the durability factors, and the results of flexural strength on both the unfrozen control bars and the bars which had been frozen and thawed. The durability factor (DF) was calculated as follows:

$$DF=\frac{(100-L)n}{70}$$

Where:

L=loss in N^2 at n cycles.

n=number of cycles at which N^2 reaches 40 percent or 70 if loss of 40 percent is not reached by end of test (70 cycles). Durability factors of 70 or greater for the particular conditions of this test are considered satisfactory.

The concrete without admixture showed a loss in N^2 of 56 percent after 6 cycles of freezing and thawing. The freezing and thawing bars had a flexural strength of 24 percent of the unfrozen control bars. The durability factor was 3.

The concretes containing two-thirds of a gallon of paraffin base oil per sack of cement and two-thirds of a gallon of asphalt base oil showed a reduction in N^2 of 53 percent and 54 percent, respectively, after 11 cycles of freezing and thawing, and the flexural strengths were 30 and 31 percent of the corresponding control bars. The durability factor was 7 for both concretes.

The bars containing one-eighth of a gallon of used oil and two-thirds of a gallon of used oil showed a loss in N^2 of 17 and 22 percent, respectively, after 70 cycles of freezing and thawing. The flexural strengths were 66 and 55 percent of that of the unfrozen specimens. However, these concretes contained 5.2 and 10.0 percent air. The durability factors for these bars were 83 and 78 representing very good resistance to freezing and thawing.

The results of the volume change tests are shown in table 11 and in figure 10. The bars for the volume change tests were made with stainless steel gauge plugs cast in the ends

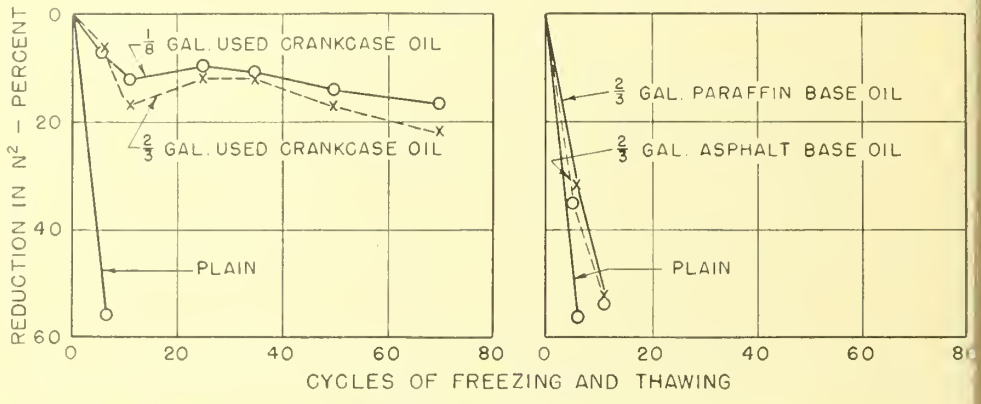

Figure 9.—Effect of admixtures of oils on the resistance of concrete to freezing and thawing.

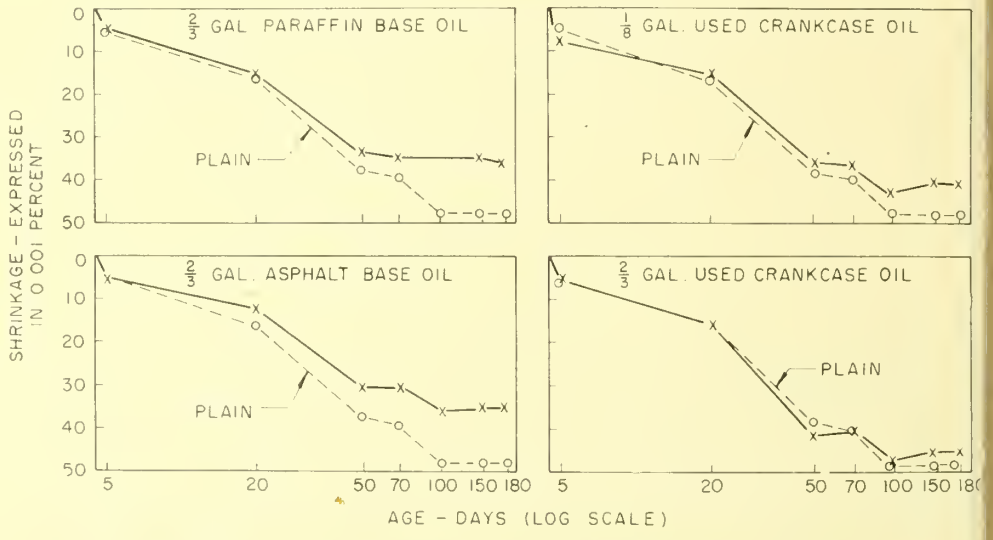

Figure 10.—Effect of admixtures of oils on the shrinkage of concrete.

Table 10.—Effect of oils on the resistance of concrete to freezing and thawing in water

Admixture		Air [2]	Loss in N^2 at—						Durability factor [3]	Flexural strength, 3- by 4- by 16-inch beams [4]	
Amount	Type		6 cycles	11 cycles	25 cycles	35 cycles	50 cycles	70 cycles		Control	Freezing and thawing
Gal./sack		*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*		*P.s.i.*	*P.s.i.*
.....	None	1.2	56	---	---	---	---	---	3	970	235(24)
2/3	Paraffin base oil	1.3	32	53	---	---	---	---	7	1,030	305(30)
2/3	Asphalt base oil	1.3	35	54	---	---	---	---	7	895	285(31)
1/8	Used crankcase oil	5.2	7	12	10	11	14	17	83	905	600(66)
2/3	do	10.0	6	17	12	12	17	22	78	740	410(55)

[1] Each value is average of 3 tests on 3 beams.
[2] Air content determined by ASTM tentative method C231-49T.
[3] Durability factor calculated at 70 cycles of freezing and thawing.
[4] Beams tested with bottom as molded in tension (4-inch depth). Figures in parentheses indicate the percentage of the strength of the corresponding unfrozen control specimens.

[3] *Evaluation of air-entraining admixtures for concrete*, by F. H. Jackson, C. E. Thorne, Proceedings ASTM, Feb 1954, vol. 27, No. 12.

[4] *Application of sonic method to freezing and thawing studies of concrete*, by F. B. Hornibrook, ASTM Bulletin No. 140, Dec 1939, p. 5.

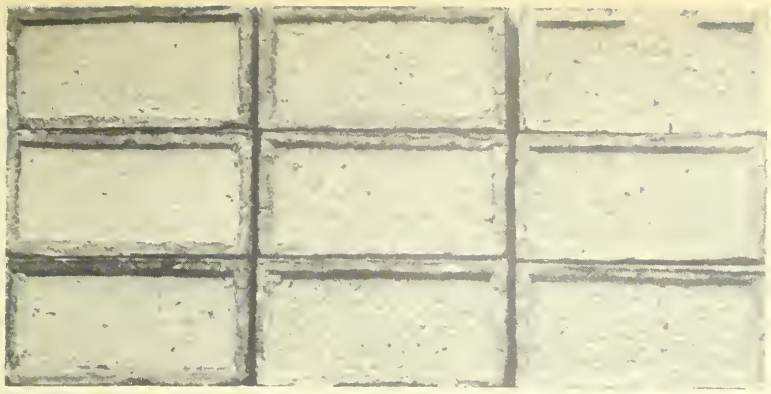

Figure 11.—Effect of thawing agent on the resistance of non-air-entrained concrete after 17 cycles of laboratory freezing and thawing.

They were stored in moist air in the molds for 2 days. After removal from the molds, they were stored in laboratory air at 72° F. and 50 percent relative humidity. The bars were measured when they were removed from the moist room and then after regular intervals of drying. The percent reduction in length was calculated from these measurements. After 180 days storage, the test was discontinued. All the concretes containing admixtures showed less shrinkage than the plain concrete.

Use of urea, part 4

Urea, an organic compound, which is reported to be non-corrosive to metals, had been suggested as a thawing agent to replace the more commonly used chlorides particularly on streets with underground streetcar cables. The tests in this investigation were made to compare the ice-melting properties of urea with calcium chloride and to study the effect of urea on the surface of non-air-entrained concrete when used for ice removal.

A comparison was made of concrete slabs on which no thawing agent was used with slabs on which either calcium chloride or urea was used. The rate of application of the thawing agent was varied from 1 to 3 pounds per square yard of exposed surface of the slab. Table 12 shows the scale ratings at 10, 15, and 17 cycles of freezing and thawing, and figure 11 shows the slabs after 17 cycles. The slabs shown in the figure were thawed with 2 pounds per square yard of either salt applied to the surface of the slabs.

In general, the rate of application of the thawing agent appeared to have only a slight effect on the scale resistance of the concrete, After 10 cycles, the slabs on which 1 pound of calcium chloride was used had a rating of 7 and the ones on which 2 or 3 pounds were used had ratings of 9. After 15 cycles all specimens on which calcium chloride was used had a rating of 10.

The specimens on which urea was used, in general, showed slightly less scaling than those on which calcium chloride was used. The use of smaller amounts of urea per square yard of surface caused less depth of scaling and also appeared to retard the start of scaling slightly as may be seen from the ratings in table 12.

The rate of thawing of the ice on the slabs on which urea was used appeared to be slower than the thawing rate when calcium chloride was used. The slabs on which no thawing agent was used showed only light scale after 17 cycles of freezing and thawing in the air of the laboratory.

Use of rust inhibitor

It is claimed that the corrosive effect of calcium chloride on steel such as automobile fenders can be greatly retarded by inhibiting the action with buffer materials. Two different percentages of an inhibitor were mixed with calcium chloride and applied to the surface of the concrete slabs. The mixture containing 1 percent inhibitor by weight of the calcium chloride was used at three different rates of application, 1, 2, and 3 pounds per square yard, and the mixture containing 5 percent inhibitor was used only at the rate of 1 pound per square yard of surface.

Table 12.—Effect of urea and inhibitor for calcium chloride on the resistance of concrete slabs to scaling, part 4 of laboratory tests [1]

Rate of application and thawing agent	Rating after freezing and thawing for—		
	10 cycles	15 cycles	17 cycles
None	2	3	3
1 lb. per sq. yd.:			
$CaCl_2$	7	10	10
$CaCl_2$+1 % inhibitor	7	8	8
$CaCl_2$+5 % inhibitor	7	8	8
Urea	5	8	8
2 lb. per sq. yd.:			
$CaCl_2$	9	10	10
$CaCl_2$+1 % inhibitor	8	9	10
Urea	6	8	9
3 lb. per sq. yd.:			
$CaCl_2$	9	10	10
$CaCl_2$+1 % inhibitor	10	10	10
Urea	8	9	10

[1] Each value is the average of 3 tests. Slabs cured in moist air 14 days and then laboratory air for 90 days. Concrete was non-air-entrained.

In table 12 are shown the relative scaling ratings determined after 10, 15, and 17 cycles of freezing and thawing. It will be noted that the rust inhibitor had little effect on the scaling action of the concrete caused by calcium chloride and its use as a rust inhibitor with calcium chloride would have no practical significance as far as the resistance of the concrete is concerned. There appeared to be no difference in the extent of scaling on the slabs between those on which mixtures of 1 percent and 5 percent inhibitor were used with the thawing agent.

Vacuum treatment, part 5

It has been demonstrated that the use of vacuum mats consolidates plastic concrete with the consolidation probably being greater at the surface than in the body of the concrete.

Vacuum treatment was applied to two types of concrete, air-entrained and non-air-entrained. Two cement contents, 6 and 7 sacks per cubic yard, were used with and without air. The details of the mixes and the air contents are shown in table 2. The method of using simulated the commercial method of application to flat slabs using a vacuum pad and pump. The scale ratings are given in table 13.

The non-air-entrained concrete containing 6 sacks of cement per cubic yard and subjected to a vacuum treatment showed a much improved surface resistance to the action of the chloride as compared to the untreated concrete. The slabs made from concrete containing 6 sacks of cement, non-air-entrained, and untreated were rated 10 at 30 cycles, whereas those with the vacuum treatment were rated only 4 after 55 cycles. Photographs of the slabs for the concrete containing 6 sacks of cement per cubic yard are shown in figure 12 and for the 7-sack concrete in figure 13.

In the case of the concrete containing 7 sacks of cement without entrained air, the improvement was very much less than that

Table 11.—Effect of oils on the drying shrinkage of concrete [1]

Admixture		Air [2]	Reduction in length (0.001 percent) after storage in laboratory air at 72° F. and 50 percent relative humidity for—						
Amount	Type		5 days	20 days	50 days	70 days	100 days	150 days	180 days
Gal./sack		*Pct.*							
	None	1.2	6	17	38	40	48	48	48
2⅔	Paraffin base oil	1.3	6	16	34	35	36	35	36
2⅔	Asphalt base oil	1.3	6	12	31	31	36	35	35
1⅓	Used crankcase oil	5.2	8	16	37	37	43	41	41
2⅔	do	10.0	4	17	40	40	47	44	44

[1] Each value is average of tests of 3 beams.
[2] Air content determined by ASTM tentative method C231–49T.

observed for similar concrete containing 6 sacks of cement per cubic yard. The vacuum-treated slabs were tested at 30 cycles and 10 at 55 cycles compared to the 6-sack concrete tested at 30 cycles and 1 at 55 cycles. This is in agreement with other tests of vacuum treated concrete. It has been observed that the leaner mixes are compacted more because of the greater quantity of water removed.

It was found that concretes with air entrainment and containing both 6 and 7 sacks of cement per cubic yard were greatly improved in resistance to the chloride attack by use of the vacuum method. This was not anticipated as it was believed that the vacuum treatment would not benefit air-entrained concrete because such concrete has inherent resistance in the first place, and secondly the vacuum treatment would lower the air content at the surface. The only explanation that appears reasonable is the probable reduction in the water-cement ratio at the surface of the concrete.

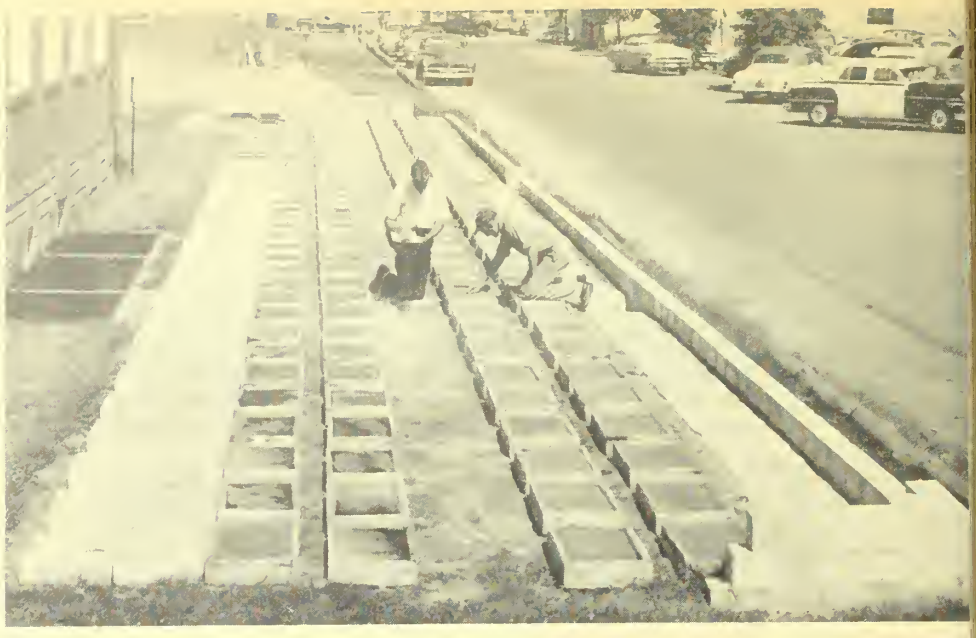

Figure 11.—Exposure area for test specimens

Discussion of Outdoor Tests

As mentioned earlier in this report, the concrete specimens for the outdoor exposure test were 16 by 24 by 4 inches deep. The concrete contained approximately 6 bags of cement per cubic yard and the maximum size of aggrega was 1 inch. Further details of the concre proportions are given in table 14. Tv cements were used, a high and a low alk cement; table 1 shows the chemical analys of the cements.

The specimens exposed outdoors during tl winter of 1951–52 were cast in water-tig wooden molds with metal bottoms. Tl concrete specimens were made in the la oratory during the spring of 1951, and cur in the moist room until the summer of 195 They were then placed outdoors and in tl fall when freezing was expected, the surfac were covered with from one-fourth to one-ha inch of water. The water was held on tl surface by the raised edge on top of the spec men. On those slabs tested during the wint of 1951–52, 19 cycles of freezing and thawi were obtained. Any freezings that occurr on weekends are not included in the tes because in general no salt application w made on either Saturday or Sunday duri this series.

Figure 14 shows the exposure plot phot graphed in the spring of 1954. The meth of examining and removing any loose mort before rating the slab is shown on the cov page. The same scale of rating was used f the large slabs as for the small laborato slabs. A rating of 5 or more was considere major scaling.

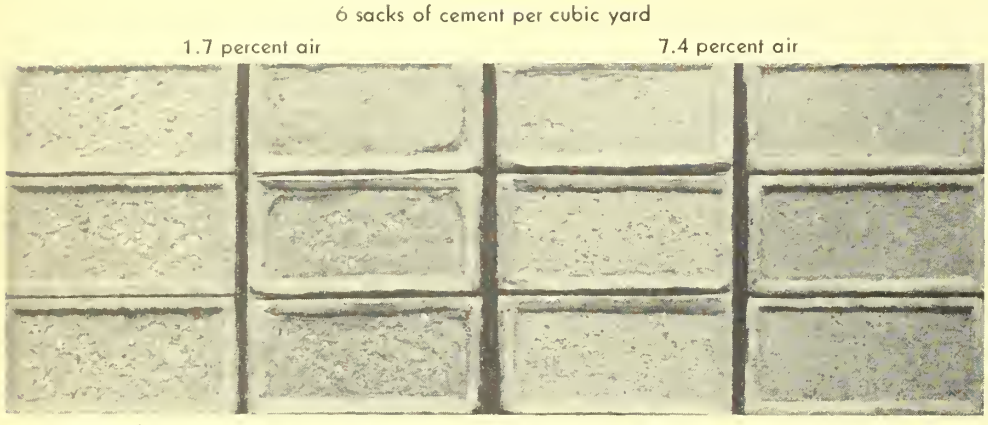

Figure 12.—Effect of vacuum-surface treatment on resistance to scaling of concrete after indicated cycles of laboratory freezing and thawing with calcium chloride.

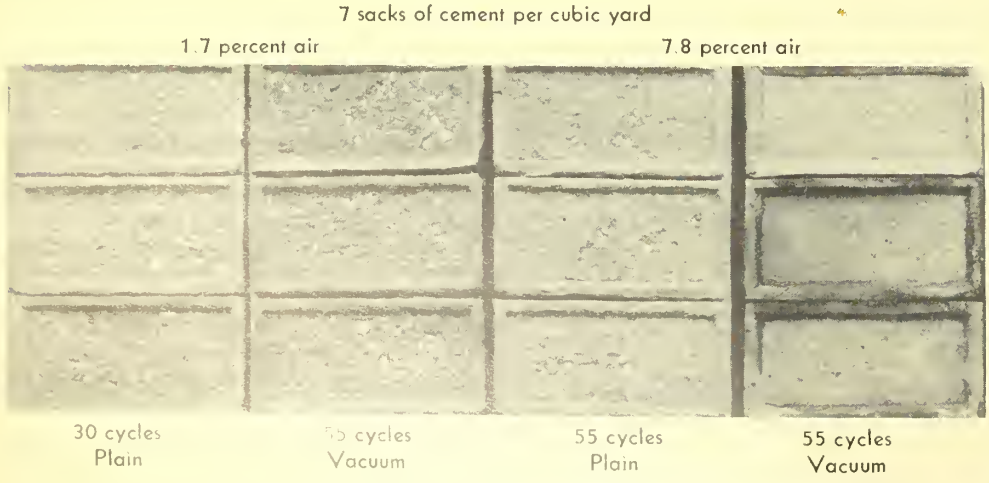

Figure 13.—Effect of vacuum-surface treatment on resistance to scaling of concrete after indicated cycles of laboratory freezing and thawing with calcium chloride.

Air-entraining admixture, part 1

One slab was made for each admixtu tested. As a basis of comparison two no air-entrained slabs were made. The thawin agent used was calcium chloride and it w applied at the rate of 2.4 pounds per squa yard in a manner similar to that used on t laboratory specimens.

Table 15 shows the ratings of the sla after 7, 12, and 19 cycles. The differe admixtures have been arranged in the tab in seven groups corresponding to the groupin in the evaluation tests of air-entraining a

Table 13.—Effect of vacuum treatment on the resistance of concrete to scaling, part 5 of laboratory tests [1]

Cement	Air [2]	Surface treatment	Rating after freezing and thawing for—			
			15 cycles	25 cycles	30 cycles	55 cycles
Sack/cu. yd.	*Percent*					
6.0	1.7	None	3	9	10	--
6.0	1.7	Vacuum	2	3	3	4
6.2	7.4	None	0	2	3	5
6.2	7.4	Vacuum	1	1	1	2
7.0	1.7	None	3	9	10	--
7.0	1.7	Vacuum	3	8	8	10
7.1	7.8	None	1	3	3	5
7.1	7.8	Vacuum	0	0	0	0

[1] Each value is average of 3 tests. Slabs stored in moist air 14 days followed by 21 days in laboratory air.
[2] Air content determined by ASTM tentative method C231–49T.

mixtures described in a previous report and with the same identification numbers.[5]

The non-air-entrained concretes used as a basis of comparison had light scale over most of the surface and some moderately deep scale at 7 cycles (ratings of 5 and 6) and both were badly scaled (rating of 10) at the end of 19 cycles.

With one exception, the use of air-entraining admixtures consisting of salts of wood resin resulted in concretes having very good resistance to the action of calcium chloride. The one exception, admixture No. 6, was rated 6 at the end of 19 cycles.

The use of the air-entraining admixtures, consisting of synthetic detergents, in concrete was ineffective in reducing scaling. The concretes had poor resistance with all but two showing complete scaling of the surface (rating of 10) at 19 cycles. Even the two exceptions were rated 4 and 6 at 19 cycles.

Admixtures of the sulfonated lignin type were effective in preventing major scaling up to 19 cycles at which time the tests were discontinued. All specimens were rated 1 at 19 cycles.

There were only two admixtures in the group of salts of petroleum acids. One of these in concrete had good resistance to scaling and the other admixture in this group was fair (rating of 6 at 19 cycles).

Neither of the two admixtures made from salts of proteinaceous acids had much value in reducing scaling of concrete even though these admixtures entrained air comparable with the other air-entraining admixtures. At the end of 19 cycles the surfaces of the concretes were nearly as bad as those concretes containing no air-entraining admixtures. However, major scaling was delayed to 10 or 12 cycles.

The four admixtures consisting of fatty and resinous acids and their salts were very effective and all the concretes in which they were used showed excellent resistance to calcium chloride attack. Likewise the admixture containing organic salts of sulfonated hydrocarbons was of value in concrete in preventing scaling caused by the surface application of calcium chloride.

These tests are of particular interest because they show that the percentage of entrained air alone may not be the controlling factor in determining the degree of resistance to calcium chloride attack. Some of the concretes having air contents below 4.5 percent showed good resistance and some containing air contents over 5 percent showed rather poor resistance.

After the 19 cycles of freezing and thawing with calcium chloride, the slabs were sawed into five 4- by 4- by 16-inch beams. These beams were tested for flexural strength on a 12-inch span with center-point loading. The tops as molded (the surface on which calcium chloride was used) were in tension. The strengths were relatively high with only one value below 700 p.s.i. modulus of rupture which indicates that the concrete under the scale was structurally sound.

Other Variables Considered

Because a single slab was made for each condition in the preliminary investigation, it

[5] See footnote 3, p. 150.

Table 14.—Mix data for slabs exposed outdoors, (16 by 24 by 4 inches deep) [1]

Concrete	Mix by dry weight	Cement	Water	Slump	Air
	Pounds	*Sack/cu. yd.*	*Gal./sack*	*Inches*	*Percent*
Non-air-entrained	94–215–315	6.0	6.1	3.0	1.8
Air-entrained	94–200–315	6.0	5.6	3.0	6.0

[1] Figures given are average values for parts 1 to 4 inclusive. In part 2, 33⅓ percent of the cement was replaced by equal solid volume of fly ash. Materials used: Type I cement, brands A and B; siliceous sand, F. M.=2.70; crushed limestone coarse aggregate, 1-inch maximum size.

Table 15.—Effect of air-entraining admixtures on resistance of concrete to scaling, preliminary tests of cement brand A molded on a metal base and exposed outdoors [1]

Admixture number	Air	Rating after freezing and thawing for—			Modulus of rupture [2]
		7 cycles	12 cycles	19 cycles	
BASE MIX: NO AIR-ENTRAINING ADMIXTURE					
	Percent				*P.s.i.*
None	1.1	6	8	10	905
Do.	1.3	5	7	10	755
SALTS OF WOOD RESIN					
1	5.5	1	1	2	830
2	6.2	1	1	1	710
3	6.4	1	2	2	650
4	5.2	1	2	2	810
5	5.6	0	1	3	775
6	5.0	1	4	6	800
7	4.5	0	1	2	830
SYNTHETIC DETERGENTS					
12	6.4	1	3	6	705
13	5.0	1	2	4	780
14	4.0	4	8	10	815
15	4.8	3	8	10	770
16	4.8	3	7	10	750
17	4.8	2	7	10	750
26	5.4	1	6	10	790
SALTS OF SULFONATED LIGNIN					
18	5.8	0	1	1	940
20	6.7	0	1	1	870
21	6.4	0	1	1	935
22	3.2	0	1	1	1080
SALTS OF PETROLEUM ACIDS					
11	6.5	0	0	1	790
27	5.0	2	4	6	860
SALTS OF PROTEINACEOUS MATERIALS					
24	5.1	3	5	8	790
25	5.6	2	6	9	750
FATTY AND RESINOUS ACIDS AND THEIR SALTS					
8	4.2	1	1	1	860
9	5.6	0	1	1	815
10	4.8	0	1	2	810
28	6.4	1	1	2	825
ORGANIC SALTS OF SULFONATED HYDROCARBONS					
23	5.3	0	1	1	810

[1] Slabs made Feb. 1951, stored in moist air about 100 days. Outdoor freezing and thawing with $CaCl_2$ for 19 cycles during the winter of 1951–52.
[2] After 19 cycles of freezing and thawing each slab was sawed into five 4- by 4- by 16-inch beams. The beams were tested by center loading on a 12-inch span with the top as molded in tension.

Table 16.—Effect of air-entraining admixtures on resistance of concrete to scaling, part 1, cements A and B molded on a metal base and exposed outdoors [1]

Admixture No.	Cement A: Air	Cement A: Rating after freezing and thawing for— 12 cycles	17 cycles	39 cycles	51 cycles	Cement B: Air	Cement B: Rating after freezing and thawing for— 12 cycles	17 cycles	39 cycles	51 cycles
Base Mix: No Air-Entraining Admixture										
None	*Percent* 1.8	6	10	10	10	*Percent* 1.0	4	8	10	10
Do.	2.1	1	5	10	10	2.1	8	10	10	10
Do.	1.8	8	10	10	10	1.1	8	10	10	10
Do.	1.9	6	10	10	10	1.1	8	10	10	10
Do.	1.0	8	10	10	10	0.6	3	10	10	10
Salts of Wood Resin										
1	6.1	1	1	1	2	5.7	0	1	1	1
2	7.3	1	1	3	6	5.8	1	1	2	3
3	6.0	1	1	2	2	6.5	1	1	2	2
4	6.1	1	3	6	7	6.0	1	1	2	3
6	4.9	1	1	1	2	6.6	1	1	2	2
7	6.3	1	1	1	1	5.1	1	1	2	3
30	5.7	1	1	1	1	7.8	2	2	2	2
31	4.0	1	2	2	3	5.6	1	2	2	3
32	6.4	0	1	1	1	6.4	1	1	1	2
Synthetic Detergents										
12	6.7	1	2	2	4	5.6	0	1	1	2
14	5.4	1	1	2	3	4.6	1	2	2	2
17	6.3	0	1	3	5	4.9	1	5	7	8
26	8.4	0	1	1	1	4.8	1	1	1	1
26 [2]	5.6	1	1	7	9	-----	----	----	----	----
Salts of Sulfonated Lignin										
18	7.0	1	1	1	2	5.5	1	1	1	1
19	3.7	1	1	1	2	2.8	2	4	5	7
21	5.1	1	1	1	2	5.5	0	1	2	2
Salts of Petroleum Acids										
11	7.4	0	1	2	4	6.7	1	1	1	1
27	6.5	1	1	1	1	4.1	1	1	1	3
27 [2]	-----	----	----	----	----	5.1	1	1	2	2
Salts of Proteinaceous Materials										
24	5.5	1	1	4	9	4.3	1	2	4	8
25	6.3	0	1	3	6	4.1	1	1	2	6
Fatty and Resinous Acids and Their Salts										
8	7.6	0	1	1	3	8.4	1	1	1	1
8 [2]	-----	----	----	----	----	4.3	1	1	1	5
9	6.8	1	1	1	2	5.6	1	1	2	2
10	6.7	0	1	1	2	5.5	1	1	1	2
Organic Salts of Sulfonated Hydrocarbons										
23	7.5	1	1	1	2	5.6	1	1	1	3
Miscellaneous										
33	5.7	1	1	1	2	4.5	1	2	2	2
34	5.7	1	1	1	2	6.9	1	1	1	1

[1] Cement A slabs made Feb.–May 1952, and cement B slabs, Mar.–June 1952, then stored in moist air 30 to 120 days and placed in exposure area. Outdoor freezing and thawing with $CaCl_2$ during the winter of 1952–53 for 17 cycles and winter of 1953–54 for 34 cycles.

[2] Repeat of admixtures, 8, 26, and 27 with correct amount of air.

was considered desirable to repeat the early work and extend it to cover other variables. In the spring of 1952 most of the same 27 air-entraining admixtures and a few other admixtures received too late to be used the first year were used with each of two different portland cements to make two series of exposure slabs similar to those made the previous year. The two cements are identified as cements A and B. The winter of 1952–53 was very mild and the 17 cycles obtained were not nearly so severe as those obtained the previous winter. Under the conditions of test all the concretes regardless of the air-entraining admixture or brand of cement gave good resistance after the first year of exposure. These slabs remained in the exposure area during the summer of 1953 and were all tested again during the winter of 1953–54 and were subject to an additional 34 cycles of freezing and thawing, making a total of 51 cycles. Thirty-four cycles were obtained by making tests on Saturdays and Sundays. However, this was also a mild winter with the temperature seldom falling below 25° F.

The ratings of the slabs made with cements A and B are given in table 16. It is interesting to note that the extent of scaling at the end of 51 cycles in general was not as severe as that obtained in the earlier test in 1951–52 when the concrete was exposed to only 19 cycles of severe freezing. The two mild winters and the greater age of the slabs at the time of the second exposure probably accounted for the better resistance of the second series of tests.

Two slabs were made using admixture No. 26 and cement A. In the first the air content was 8.4 percent which was greater than intended. For this reason another slab was made with an air content of 5.6 percent. The slabs with 5.6 percent air were badly scaled (rating of 9) at 51 cycles, whereas the slab with 8.4 percent had a rating of 1 at 51 cycles. This was an indication that slight differences in air contents may account for differences in the scaling on different slabs containing the same admixture. However, when this admixture was used in concrete with cement B and the air content was only 4.8 percent, the resistance to attack by calcium chloride was very good.

A comparison of the results obtained with the two cements used indicates that the slabs containing type B cement were not as severely attacked as those made from type A. In the group of wood resins the concretes containing admixtures 2 and 4 showed ratings of 3 for type B cement at the end of 51 cycles, and the same admixture made with type A cement had scale ratings of 6 and 7, respectively. In the case of concretes containing synthetic detergents, all the slabs with cement A showed extensive scaling except the one containing 8.4 percent air, whereas with those made with cement B only one of the four slabs showed more than slight (rating 2) scaling.

The concrete slab containing cement B and admixture No. 19, sulfonated lignin, and having an air content of 2.8 percent had a rating of 7 at 51 cycles, whereas all the other slabs with higher air contents containing sulfonated lignin had ratings of 2 or less. The lack of resistance to salt action of the one slab in this group may be attributed to the low percentage of entrained air.

The concretes containing salts of petroleum acids, fatty acids and resinous acids and their salts, and the miscellaneous air-entraining admixtures showed satisfactory resistance when used with both cements.

The concretes containing salts of proteinaceous materials showed major scaling with both cements.

One or two representative air-entraining admixtures of each group were used in concrete cast on a sand base. Comparable slabs were cast for both cements A and B. The ratings of these slabs are shown in table 17. Only one slab with cement B had a rating of 3, two had a rating of 1, and the others were rated 2 at 51 cycles.

Table 18 shows a direct comparison between metal and sand bases. In general, the action

Table 17.—Effect of sand bases on resistance of air-entrained concrete to scaling, part 1, cements A and B molded on a sand base and exposed outdoors [1]

Admixture number	Cement A					Cement B				
	Air	Rating after freezing and thawing for—				Air	Rating after freezing and thawing for—			
		12 cycles	17 cycles	39 cycles	51 cycles		12 cycles	17 cycles	39 cycles	51 cycles
Base Mix: No Air-Entraining Admixture										
None	*Percent* 1.1	1	8	10	10	*Percent* 0.6	2	4	8	10
Salts of Wood Resin										
2	5.0	1	2	2	2	6.0	1	1	1	2
Synthetic Detergents										
12	3.7	1	2	2	2	4.5	1	1	1	2
15	6.7	1	2	2	2	4.8	1	1	1	1
Salts of Sulfonated Lignin										
19	3.5	1	1	2	2	2.6	1	1	2	3
Salts of Petroleum Acids										
27	4.3	1	2	2	3	4.3	1	1	1	2
Salts of Proteinaceous Materials										
24	4.5	1	2	2	3	3.7	1	1	2	2
Fatty and Resinous Acids and Their Salts										
10	4.3	1	2	2	2	4.2	1	2	2	2
Organic Salts of Sulfonated Hydrocarbons										
23	5.3	1	2	2	3	6.1	1	1	1	1

[1] Slabs made in June 1952, stored in moist air for 30 days, and then stored in exposure area. Outdoor freezing and thawing with $CaCl_2$ during the winter of 1952–53 for 17 cycles and during the winter of 1953–54 for 34 cycles.

Table 18.—Comparison of metal and sand bases on the resistance of air-entrained concrete to scaling, part 1 of outdoor tests [1]

Admixture number	Rating at 51 cycles			
	Cement A		Cement B	
	Metal base	Sand base	Metal base	Sand base
None	10	10	10	10
2	6	2	3	2
12	4	2	2	2
19	2	2	7	3
27	1	3	2	2
24	9	3	8	2
10	2	2	2	2
23	2	3	3	1

[1] These data also appear in tables 16 17.

of calcium chloride is less severe on air-entrained concrete that is cast on a sand base than when the concrete is cast in a watertight metal based mold that does not allow any of the water to escape from the plastic concrete.

Use of fly ash, part 2

Four fly ashes, a fine and coarse fly ash from each of two sources, were each used to replace 33⅓ percent of the cement in a 6-sack mix. The fly ashes (A and B, table 19) from one source had carbon contents of less than 1 percent and those from the other source (X and Y, table 19) had carbon contents of 5.0 and 11.2 percent. From each source the coarser material had the lower carbon content. Two cements were used, cement A and cement B. Cement B had a very low alkali content and cement A a high alkali content. Table 1 contains a comparison of the chemical analyses and calculated compound composition.

The concrete for the exposure slabs was cast in molds with metal bases. Two types of

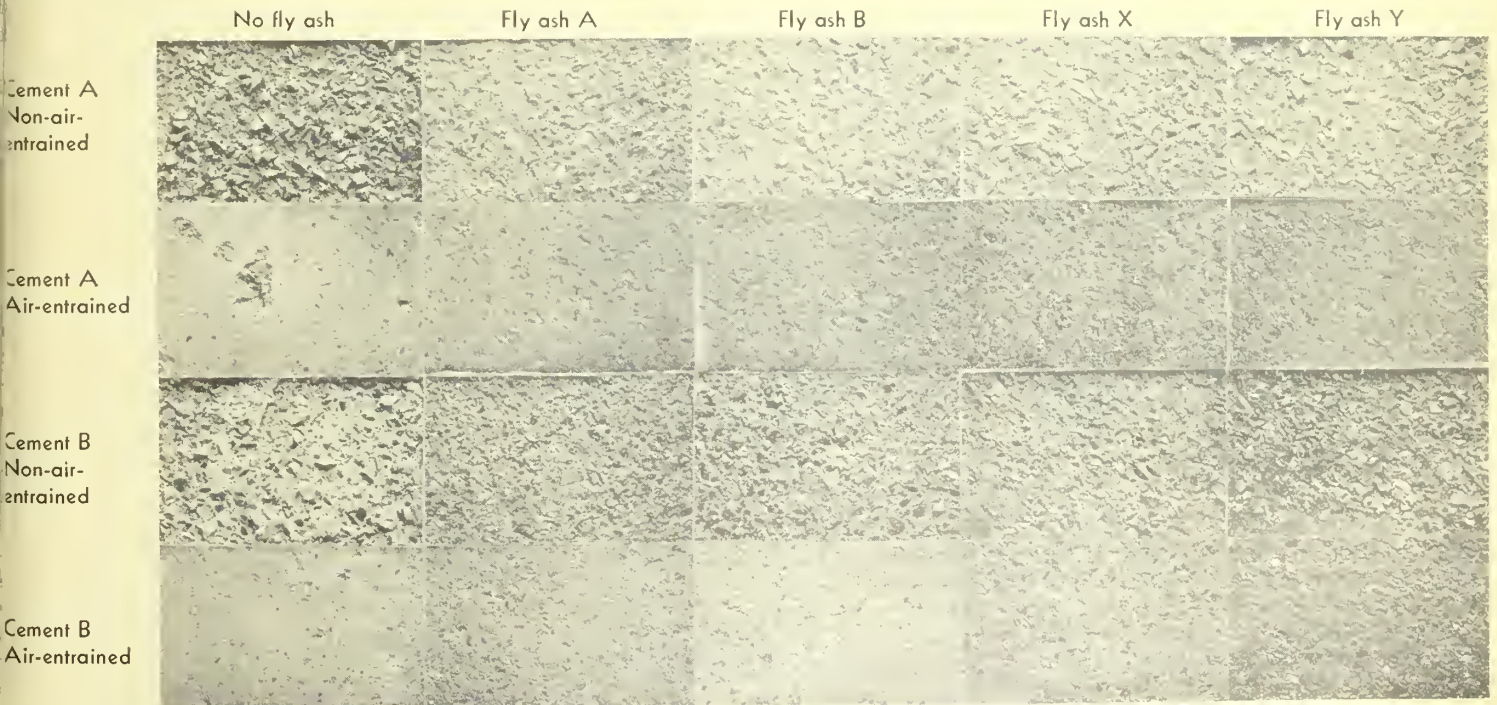

Figure 15.—Effect of fly ash on resistance of concrete to scaling. Test panels photographed in 1954 after 51 cycles of outdoor freezing and thawing with calcium chloride.

Table 19.—Effect of fly ash on resistance of concrete to scaling, part 2, cements A and B molded on metal bases and exposed outdoors [1]

Cement	Fly ash [2]	Air [3]	Rating after freezing and thawing for— 12 cycles	17 cycles	39 cycles	51 cycles
			Non-Air-Entrained Concrete: Cement A			
		Percent				
A	None	1.0	5	8	10	10
A	A	1.0	6	8	10	10
A	B	1.0	6	8	10	10
A	X	1.0	7	8	10	10
A	Y	1.0	6	8	10	10
			Air-Entrained Concrete: Cement A			
A	None	4.0	1	2	2	3
A	A	4.6	2	3	3	4
A	B	5.4	3	4	4	4
A	X	4.9	4	6	7	7
A	Y	4.3	4	6	6	7
			Non-Air-Entrained Concrete: Cement B			
B	None	1.0	3	8	10	10
B	A	1.0	4	8	9	10
B	B	1.0	4	8	10	10
B	X	1.0	6	8	10	10
B	Y	1.0	5	8	10	10
			Air-Entrained Concrete: Cement B			
B	None	6.8	1	2	2	2
B	A	4.0	1	2	4	4
B	B	3.3	5	6	7	7
B	X	3.9	6	8	8	8
B	Y	5.8	6	8	8	8

[1] Slabs made June 1952, stored in moist air 30 days, then stored in exposure area. Outdoor freezing and thawing with $CaCl_2$ during the winter of 1952–53 for 17 cycles and during the winter of 1953–54 for 34 cycles.
[2] Where fly asb was used, 33⅓ percent of the cement was replaced by an equal volume of fly ash.
[3] Air in non-air-entrained mixes calculated, others measured by ASTM tentative method C231–49T.

concrete were used, non-air-entrained and air-entrained concrete. The ratings at 12, 17, 39, and 51 cycles are shown in table 19, and photographs at 51 cycles are shown in figure 15.

The plain concretes without air-entrainment and without fly ash replacements for part of the cement had very poor resistance to attack by the chloride salt. None of the fly ashes used as replacements for part of the cement were effectual in improving the resistance of non-air-entrained concrete to attack by calcium chloride.

Entrained air greatly increased the resistance of the plain concrete. In air-entrained concrete all the fly ash replacements for cement were detrimental to the resistance to scaling of the concrete, as indicated by the ratings of 4 to 8. The alkali content of the cement appeared to have had no relation to the resistance of the resulting concrete to attack by calcium chloride used for ice removal. The extent of attack by calcium chloride did not appear to differ much with the brand of cement or with the fly ash used as a replacement for cement.

Type of curing, part 3

Table 20 gives the ratings of concrete slabs cured by different methods. Portland cement A was used in all slabs. The slabs were cast outdoors in wood molds on sand bases and remained in the molds for 3 days. They were then removed from the molds, the sides painted, and the slabs placed in the exposure area. Two rounds of slabs were made for the non-air-entrained concrete on different days, and one round for the air-entrained concrete. For purposes of comparison, slabs given no curing were cast in molds with metal bases and with sand bases.

Considerable difference in resistance was observed between the two rounds of test slabs regardless of curing treatment. All the slabs in round 1, even the slab given no curing, showed better resistance than those cast in round 2 except burlap plus oil. This difference in rounds is a common experience in curing studies carried out under the humidity conditions which prevail in the Washington area. Because of the greater attack on the slabs in round 2, differences due to curing are more apparent. In the discussion that follows only the second round is considered.

In general, the different methods of curing had little effect on the scale resistance of the resulting concrete. Wet burlap curing for 3 days followed by a lubricating oil surface treatment at 28 days was effective in reducing scaling. Concrete containing an admixture of lubricating oil and cured 3 days under wet burlap was also effective in reducing scaling.

With air-entrained concrete little or no scaling occurred, and it is not possible to distinguish between the relative effect of any of the curing methods tried.

Vacuum treatment, part 4

Comparisons were made between the regular method of finishing concrete and the vacuum method. The ratings of the slabs after various cycles are shown in table 21. These specimens were made and cured outdoors. After curing under wet burlap for 3 days the sides were waterproofed and the slabs were then placed in the exposure area.

Non-air-entrained concrete was cast in molds with metal bases and in molds with

Table 20.—Effect of curing on resistance of concrete to scaling, part 3, cement A molded on bases indicated and exposed outdoors [1]

Type base	Initial curing [2]	Surface treatment [3]	Admixture	Rating after freezing and thawing for— 12 cycles	17 cycles	39 cycles	51 cycles
			Non-Air-Entrained Concrete [4]				
Metal	None	None	None	1	1	1	1
Do	do	do	do	1	2	4	6
Sand	do	do	do	0	1	1	1
Do	do	do	do	1	2	2	3
Do	Burlap	do	do	0	1	1	1
Do	do	do	do	1	3	3	4
Do	do	Lubricating oil	do	0	1	1	1
Do	do	do	do	0	1	1	1
Do	do	None	Lubricating oil	0	1	1	1
Do	do	do	do	1	1	2	2
Do	Paper	do	None	0	1	1	1
Do	do	do	do	1	2	4	9
Do	do	Lubricating oil	do	0	1	3	5
Do	Membrane A	None	do	1	1	1	1
Do	do	do	do	0	1	1	4
Do	Membrane B	do	do	1	1	1	1
Do	do	do	do	0	1	1	2
Do	Lubricating oil	do	Lubricating oil	0	1	2	2
Do	do	do	do	1	1	1	5
Do	do	do	do	0	1	2	3
			Air-Entrained Concrete [5]				
Metal	None	None	Vinsol resin	0	1	1	2
Sand	do	do	do	0	1	1	1
Do	Burlap	do	do	1	2	2	2
Do	do	do	Used crankcase oil	1	1	1	1
Do	Paper	do	Vinsol resin	0	1	1	1
Do	Membrane A	do	do	0	1	1	2
Do	Membrane B	do	do	0	1	2	2
Do	Used crankcase oil	do	Used crankcase oil	0	0	2	2

[1] All slabs made outdoors in July 1952, removed from molds after 3 days, and then stored in exposure area. Outdoor freezing and thawing with $CaCl_2$ during the winter of 1952–53 for 17 cycles and the winter of 1953–54 for 34 cycles.
[2] Curing applied after 1½ hours of placing of slab and if removed, removed after 3 days.
[3] Surface protective treatment applied after 28 days.
[4] Air content of non-air-entrained concrete approximately 2 percent.
[5] Air content of air-entrained concrete approximately 4½ percent.

ble 21.—Effect of vacuum surface treatment on resistance of plain and air-entrained concrete to scaling, part 4, cement A molded on bases indicated and exposed outdoors [1]

Air	Type base	Surface treatment [2]	Rating after freezing and thawing for—			
			12 cycles	17 cycles	39 cycles	51 cycles
Percent						
1.0	Metal	None	1	1	8	10
1.0	do	do	1	4	10	10
1.0	do	Vacuum	1	2	4	6
1.0	do	do	1	4	7	10
1.0	Sand	None	1	1	9	10
1.0	do	do	1	4	10	10
1.0	do	Vacuum	1	1	2	4
1.0	do	do	0	1	2	3
2.6	do	None	1	1	2	4
2.6	do	Vacuum	0	1	2	2
3.2	do	None	1	1	2	2
3.2	do	Vacuum	0	1	1	1
5.2	do	None	0	1	2	2
5.2	do	Vacuum	0	1	1	1
6.1	do	None	1	1	2	2
6.1	do	Vacuum	0	1	1	1
7.0	do	None	1	1	3	3
7.0	do	Vacuum	0	1	1	2
10.0	do	None	0	1	1	1
10.0	do	Vacuum	0	0	0	0

[1] Slabs made outdoors in September 1952, cured with wet burlap for 3 days, and then stored in exposure area. Outdoor zing and thawing with $CaCl_2$ during the winter of 1952–53 for 17 cycles and the winter of 1953–54 for 34 cycles.
[2] Vacuum applied to surface of plastic concrete for ½ hour.

mp sand bases. One-half of the specimens st in each type base were finished in the reg-ır manner by brooming and the other half re subjected to vacuum finishing using a vacuum pad covering the entire surface of the concrete. A vacuum of 18 to 25 inches of mercury was applied for about 30 minutes. The vacuum pad was removed and the specimen given a final trowel finish. The [illegible] placed slabs were covered with wet burlap 1 hour after they were cast and the others were covered after 2 hours.

The concrete cast in molds with metal bases and subjected to vacuum treatment had about the same resistance to scaling as that placed by the conventional methods. One of the two vacuum-placed specimens had a little better resistance than the other. In the case of the non-air-entrained concrete cast in molds with sand bases, there was a marked improvement in the resistance to scaling of the vacuum-placed specimens. It seems likely that the metal base mold inhibits the removal of the water normally withdrawn by the vacuum process.

The air-entrained concrete specimens for vacuum treatment were cast on sand bases. The air content varied from 2.6 to over 10 percent. The lowest air content of 2.6 percent for the untreated concrete had a rating of 4 at 51 cycles while the comparable vacuum-treated concrete had a rating of 2. The untreated concrete with an air content of 10+ percent had a rating of 1 and the corresponding treated concrete had a rating of 0. In every case the vacuum-placed concrete had a slightly greater resistance to the action of calcium chloride than its untreated counterpart.

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(3) *Salt and phosphate to fight snow problems,* American City, Sept. 1952, vol. 67, No. 9, p. 108. (Article describes use of Banox, a phosphate rust inhibitor.)

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New Publication

The Bureau's *Highway Statistics 1954*, the ninth of the bulletin series presenting annual statistical and [illegible] tables of general interest on the subjects of motor fuel, motor vehicles, highway user taxation, financing of highways, [illegible] highway mileage is now available.

The 142 page publication may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1.00 a copy. The full series of the annual bulletins are available from the Superintendent of Documents, as indicated on the inside back cover of PUBLIC ROADS.

Traffic Article Postponed

The article *Trends in Traffic Volumes, Vehicle Types, and Weights* which has appeared annually in PUBLIC ROADS magazine since 1946 will not be included this year.

The comprehensive study of highways, begun in 1954 in accordance with section 13 of the Federal-aid Highway Act of 1954, was given preference over the work of reporting and analyzing the 1953 traffic trends data. The consequent postponement of this work has delayed the publishing of the usual traffic trends article for this one year. Tabular material, which would have been a part of the report had it been published, is available to subscribers of PUBLIC ROADS, and a set of tables giving 1953 traffic information will be furnished upon request.

Traffic data furnished in conjunction with the Section 13 study will undoubtedly result in revisions being made in tables now being made available for 1953. Furthermore, the new information resulting from the Section 13 study will make it possible to check a series of estimates of total rural and urban travel. These estimates have not been published since 1948, because the many circumstances affecting travel made it inadvisable to publish the information without a sufficient body of current basic data to check the estimates of rural and urban travel prepared on a trend basis.

The present plan is to publish in PUBLIC ROADS the 1953 traffic trends data along with that for 1954 when it becomes available. It is expected that the consolidated article will include an analysis of all rural and urban travel, similar to that reported in 1948.

Errata

The new pamphlet *Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways*, announced in the February issue of PUBLIC ROADS on page 141 and included with the list of publications on the inside back cover of the magazine, was incorrectly priced. The correct amount for the Manual (including the 1954 revision supplement) is $1.00 instead of 90 cents. The price of the Revisions pamphlet alone is 15 cents.

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Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.
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Selected Bibliography on Highway Finance (1951). 60 cents.
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(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$15,053	$13,116	$7,035	337.5	$2,308	$1,494	9.3	$43,649	$21,859	665.1	$59,073	$30,388	1,011.9
Arizona	5,301	7,194	4,814	93.7	1,159	823	25.3	8,360	5,821	139.3	16,713	11,458	258.3
Arkansas	11,022	10,932	5,996	429.6	1,924	1,122	29.8	17,112	8,458	423.5	29,968	15,576	882.9
California	8,687	23,260	12,881	161.2	17,301	9,284	38.6	117,811	58,098	333.7	158,372	80,263	533.5
Colorado	14,182	7,109	4,105	104.4	3,243	1,771	62.8	13,950	7,758	164.8	24,302	13,634	332.0
Connecticut	16,170	537	268	1.7	2,822	1,401	3.2	10,080	4,840	8.1	13,439	6,509	13.0
Delaware	4,523	767	389	4.0	2,064	1,032	7.0	6,866	3,793	23.9	9,697	5,214	34.9
Florida	10,604	20,407	10,425	242.0	10,303	5,260	61.6	20,118	10,345	316.9	50,828	26,030	620.5
Georgia	18,615	12,504	6,329	182.1	10,748	4,590	45.8	52,654	25,197	1,000.0	75,906	36,116	1,227.9
Idaho	3,863	8,635	5,516	112.9	3,427	2,139	61.1	11,940	7,506	186.1	24,002	15,161	360.1
Illinois	15,239	66,585	36,654	678.2	16,657	8,747	116.3	64,405	33,817	298.2	147,647	79,218	1,092.7
Indiana	18,930	34,874	17,657	144.6	22,949	11,692	133.8	28,397	15,203	111.9	86,220	44,552	390.3
Iowa	11,451	15,680	8,399	580.8	10,282	5,614	168.9	15,965	8,928	654.8	41,927	22,941	1,404.5
Kansas	14,989	9,321	4,757	692.8	3,639	1,855	62.7	19,989	9,996	881.9	32,949	16,608	1,637.4
Kentucky	12,956	12,902	6,893	89.3	4,537	2,393	75.7	26,719	13,407	254.0	44,158	22,693	419.0
Louisiana	12,884	13,772	6,875	156.8	8,463	4,231	42.7	26,245	11,960	117.4	48,480	23,066	316.9
Maine	5,939	6,861	3,738	31.8	559	283	4.5	14,942	7,574	128.8	22,362	11,595	165.1
Maryland	6,604	22,663	11,807	65.2	5,398	2,598	13.2	9,427	5,097	52.0	37,488	19,502	130.4
Massachusetts	16,409	5,809	2,894	19.2	3,494	1,740	6.4	47,845	22,347	38.8	57,148	26,981	64.4
Michigan	16,755	40,783	21,175	626.3	13,915	7,004	193.5	36,977	18,658	228.2	91,675	46,837	1,048.0
Minnesota	18,128	12,030	6,285	929.0	8,055	4,096	279.7	14,659	7,948	369.2	34,744	18,329	1,577.9
Mississippi	6,888	15,421	7,521	502.3	8,153	4,241	217.6	20,284	10,436	386.3	43,858	22,198	1,106.2
Missouri	14,373	16,767	8,845	843.1	13,403	7,118	89.9	64,519	33,285	1,002.9	94,689	49,248	1,935.9
Montana	15,580	11,714	7,047	289.5	1,549	960	24.2	22,745	14,048	442.8	36,008	22,055	756.5
Nebraska	11,593	28,847	15,116	1,116.3	4,907	2,458	100.5	17,273	10,080	486.2	51,027	27,654	1,703.0
Nevada	10,548	3,965	3,379	122.0	1,706	1,355	22.6	4,727	3,870	50.7	10,398	8,604	195.3
New Hampshire	5,611	2,168	1,259	11.7	381	194	1.7	6,297	3,074	36.9	8,846	4,527	50.3
New Jersey	22,098	5,556	2,471	45.2	5,473	2,228	5.8	19,965	9,345	27.3	30,994	14,044	78.3
New Mexico	6,769	4,600	2,860	119.9	2,754	1,706	66.3	10,200	6,531	160.3	17,554	11,097	346.5
New York	45,405	46,750	24,523	84.9	17,338	8,640	7.1	196,625	91,559	292.6	260,713	124,722	384.6
North Carolina	17,081	19,912	9,922	339.2	3,932	1,903	64.7	39,841	18,554	531.4	63,685	30,379	935.3
North Dakota	8,023	11,736	5,994	1,217.9	3,478	1,762	303.2	4,916	2,456	272.5	20,130	10,212	1,793.6
Ohio	28,200	31,503	15,432	83.4	13,502	5,643	25.6	56,251	26,775	117.6	101,256	47,850	226.6
Oklahoma	21,303	6,144	3,258	206.4	10,597	5,701	124.6	20,758	10,833	310.3	37,499	19,792	641.3
Oregon	9,347	3,978	2,379	68.9	399	244	14.3	13,847	8,567	171.4	18,224	11,190	254.6
Pennsylvania	19,141	56,049	28,013	194.4	12,489	6,029	7.2	87,274	43,138	166.5	155,812	77,180	368.1
Rhode Island	3,563	5,700	2,850	16.9	3,535	1,768	14.9	7,259	3,624	18.1	16,494	8,242	49.9
South Carolina	12,234	8,506	4,665	175.4	2,062	1,112	21.3	14,290	7,497	255.0	24,858	13,274	451.7
South Dakota	5,586	18,755	10,603	978.9	2,352	1,317	155.2	6,631	3,747	336.7	27,738	15,667	1,470.8
Tennessee	14,843	18,237	9,072	419.0	9,183	4,592	102.3	30,933	13,483	340.1	58,353	27,147	861.4
Texas	39,078	16,251	9,330	224.5	11,275	6,078	185.0	75,991	40,170	1,191.3	103,517	55,578	1,600.8
Utah	6,325	5,606	4,172	131.4	2,963	2,211	68.0	3,601	2,826	21.0	12,170	9,209	220.4
Vermont	4,847	1,125	565	17.6	129	74		7,377	3,675	64.2	8,631	4,314	81.8
Virginia	16,871	10,666	5,418	171.6	4,764	2,303	61.2	19,168	9,318	207.3	34,598	17,039	440.1
Washington	11,845	9,259	5,362	177.9	3,256	1,743	35.2	18,934	9,750	142.6	31,449	16,855	355.7
West Virginia	12,075	9,889	5,026	44.1	4,857	2,443	38.2	13,053	6,551	48.6	27,799	14,020	130.9
Wisconsin	14,324	27,883	13,692	431.0	1,162	597	16.2	27,407	13,906	273.6	56,452	28,195	720.8
Wyoming	1,457	9,269	6,077	234.5	2,369	1,548	70.2	8,184	5,150	147.4	19,822	12,775	452.1
Hawaii	4,031	3,514	1,757	3.2	1,441	527	2.8	5,436	2,559	15.1	10,391	4,843	21.1
District of Columbia	5,447	5,094	2,847	8.3	2,170	1,069	1.1	13,521	6,430	1.1	21,385	10,346	10.5
Puerto Rico	[illegible]	5,755	2,504	34.0	1,792	862	5.9	13,515	6,007	41.7	21,062	9,673	81.6
TOTAL	[illegible]	767,[illegible]0	407,151	13,996.5	306,618	157,595	3,294.7	1,458,932	735,854	13,956.1	2,532,510	1,300,600	31,247.3

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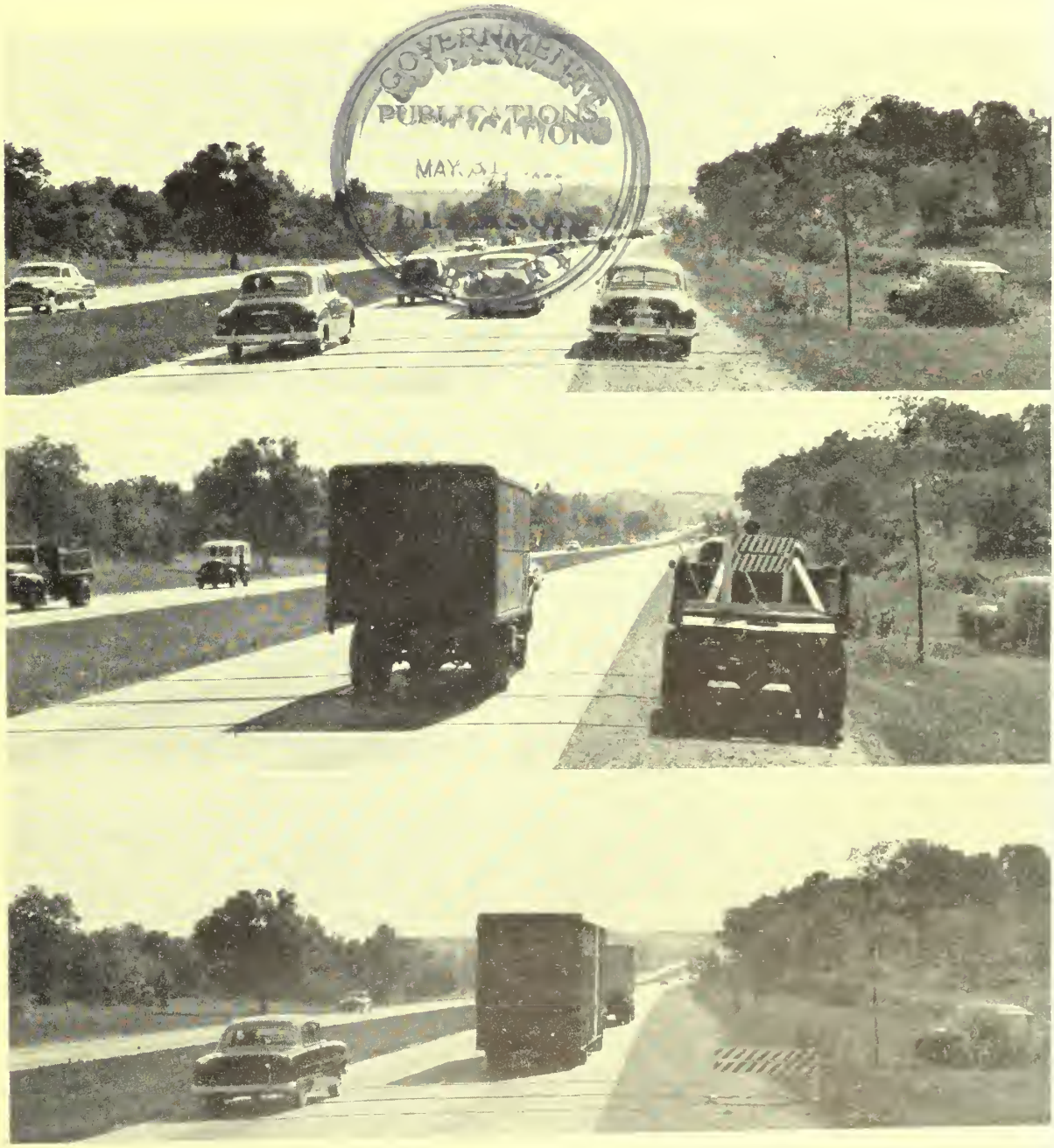

Driver reaction to objects placed on highway shoulders

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BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

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Driver Behavior as Affected by Objects on Highway Shoulders

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by ASRIEL TARAGIN
Highway Engineer

Shoulders along a highway expedite traffic and contribute to driving comfort and safety. This study was undertaken to determine how driver behavior is influenced by parked vehicles or other objects placed on shoulders at various distances from the outer edge of traffic lanes. The objects selected were a passenger car, a truck, and a barricade. Each was placed at the pavement edge, 3 feet from the edge, and 6 feet from the edge. Passenger and commercial vehicle speeds and lateral positions were recorded on two-lane highways carrying light to moderate traffic volumes and on a four-lane divided highway carrying up to 3,400 vehicles per hour in one direction.

For all vehicle types it was found that speeds were not affected significantly by objects on the shoulder. Such objects affected the lateral position of only those vehicles traveling in the adjacent lane. Objects placed at the pavement edge had the greatest effect and lateral shifts decreased sharply when objects were set 3 and 6 feet from the edge. The narrower the pavement, the further the traffic shifted left, away from the object on the shoulder. On a two-lane 16-foot highway, the shift was 3.3 feet, while on a 24-foot surface the shift was 1.8 feet. The various objects—car, truck, and barricade—caused about the same degree of shifting. On narrow two-lane roads, some vehicles shifted over the centerline when the object was placed at the pavement edge.

SHOULDERS along a highway are needed for several reasons. They add to the comfort of the drivers and increase safety. Wide shoulders provide a storage space outside the traffic lanes for disabled vehicles and for short-time parking of vehicles as may be desired by drivers. They also serve as a space to avoid an accident in case of emergency.

It is generally believed that vehicles standing along the traveled way of a highway either by choice of the drivers or due to mechanical failure influence the behavior of vehicles on the highway. It is the purpose of this article to report the results of studies made to determine the extent to which traffic is influenced by parked vehicles or other objects on the shoulders at various distances from the edges of the traffic lanes.

Vehicle speeds and placements were recorded on two-lane highways carrying light to moderate traffic and on a four-lane divided highway carrying volumes as high as 3,400 vehicles per hour in one direction.

Study Procedure

A description of the sites selected for study for two-lane operations is given in table 1. These sites were studied in cooperation with the Oregon State Highway Department and were on level tangent sections with no restrictive sight distances in either direction of travel. A yellow dashed centerline separated the two traffic lanes, and the pavements were flanked by well-maintained gravel shoulders. The 16-foot concrete pavement at site 1 was flanked by 2-foot bituminous shoulders beyond which were the 5-foot gravel shoulders.

The study of four-lane operations was conducted during September 1953, on a level tangent at the north end of the Shirley Memorial Highway, a freeway connecting the Pentagon Network and Woodbridge, Va. The section studied has two 12-foot lanes in each direction of portland cement concrete with shoulders 10 feet wide. A 30-foot grass median separates the vehicles traveling in the opposing directions. The posted speed limit at this location is 50 miles per hour for passenger cars and 45 miles per hour for trucks.

The speeds and transverse positions of vehicles were first recorded at all locations under normal conditions, that is, with the shoulders clear of any object. Similar data were then recorded with the objects placed on the shoulder at various distances from the pavement edge. The data for normal conditions were used as a base to determine the effect on driver behavior of each of the several objects placed on the shoulder. These data cannot be used, however, to compare driver behavior on various surface widths, because design elements and traffic conditions were not the same at the different locations included in this study. The results of another study based on extensive data for comparable locations show the effect of roadway width on driver behavior.[1]

[1] *Effect of roadway width on vehicle operation*, by A. Taragin, PUBLIC ROADS, vol. 24, No. 6, Oct.-Nov.-Dec. 1945.

Conclusions

From studies of driver behavior with and without objects on the shoulders of two-lane and four-lane divided highways it may be concluded that:

1. Vehicle speeds were not affected significantly by objects on the shoulder during light to moderate traffic volumes. This observation confirms earlier studies of speeds on narrow pavements and at narrow bridges and other obstructions.

2. Objects parked on one shoulder affected the lateral position of only those vehicles traveling in the lane adjacent to the occupied shoulder. The effect was greatest when the object on the shoulder was at the pavement edge, and diminished rapidly with an increase in the distance of the object from the edge of the pavement.

3. The narrower the pavement, the greater was the shift of traffic to the left or away from the object on the shoulder. On a two-lane highway with a 16-foot surface, the shift was 3.3 feet, and on one with a 24-foot surface the shift was 1.8 feet.

4. A car, truck, or barricade on the shoulder had about the same effect on the lateral positions of vehicles on two-lane highways. On the four-lane highway a truck had about the same effect as a car, but a barricade caused a lateral movement about one-half as great as a car or truck.

Objects on One Shoulder

The objects placed on the shoulder included a passenger car, a highway maintenance truck, and a barricade. Studies were conducted with each of these objects at the pavement edge, 3 feet from the edge, and 6 feet from the edge. Data were recorded during a period of 3 hours for each condition at each location. Driver behavior data were thus obtained for nine different conditions in addition to the normal condition at each study location. The type and position of the objects placed on the shoulder are shown on the cover page. The three lines across the pavement in the foreground of the photographs are the speed and placement detectors which were connected to recording instruments that were well hidden from the drivers.

The hourly traffic volume in both directions on the two-lane roads during each of the studies is shown in table 2. The table shows

Table 1.—Description of the two-lane highways studied in Oregon

Site No.	Route and location	Width of pavement [1]	Width of shoulders		Number of vehicles observed	
			Bituminous	Gravel	Day	Night
		Feet	*Feet*	*Feet*		
1	U. S. 99E, 7 miles S. of Salem	16	2	5	6,600	-----
2	U. S. 99W, S. of Tigard	20	-------	7	10,246	2,997
3	U. S. 99, 6 miles N. of Eugene	22	-------	9	9,144	1,985
4	U. S. 99E, 1 mile S. of Brooks	26	-------	5	10,694	-----

[1] Sites numbered 1 3 have portland cement concrete surfaces; site 4 has bituminous surface.

Table 2.—Hourly traffic volume observed during study periods on two-lane highways

Position of object on shoulder	Daytime				Nighttime	
	16-foot pavement	20-foot pavement	22-foot pavement	26-foot pavement	20-foot pavement	22-foot pavement
	V. p. h.	*V. p. h.*	*V. p. h.*	*V. p. h.*	*V. p. h.*	*V. p. h.*
Shoulder clear	179	233	192	282	152	-------
Car at edge of pavement	201	349	220	323	214	-------
Car 3 feet from edge of pavement	176	364	169	263	116	107
Car 6 feet from edge of pavement	196	258	213	469	107	151
Truck at edge of pavement	164	269	214	310	-------	-------
Truck 3 feet from edge of pavement	216	325	232	287	-------	-------
Truck 6 feet from edge of pavement	211	268	229	344	-------	-------
Barricade at edge of pavement	247	344	280	321	146	189
Barricade 3 feet from edge of pavement	211	307	257	394	167	99
Barricade 6 feet from edge of pavement	228	483	288	364	115	88

that traffic volumes were moderate at all loca tions and not widely different for each should condition at any one location; thus the effe of traffic volume or density was eliminate from consideration in this study.

Data were analyzed separately for passeng cars and commercial vehicles traveling in eac of the two traffic lanes. For the two-lan studies, the data were also analyzed separatel for free-moving vehicles, meeting vehicles, an all other vehicles. Free-moving vehicles ar those that were not influenced by other traffi on the highway, whereas meeting vehicles ar those that might be influenced by opposin traffic. Vehicles overtaking and passing othe vehicles on the two-lane roads have bee omitted from the analysis because they wer too small in number during the study period to be considered.

Normal Speeds

Average passenger car speeds during norma daytime conditions at the different location on the two-lane roads varied from 42.9 to 48. miles per hour as shown in table 3. Unde similar conditions, the average speeds of com mercial vehicles (dual-tired only) varied fro 37.6 to 43.2 miles per hour as shown in table 4 At individual locations, however, commercia vehicles traveled, on an average, 3.9 to 8.

Table 3.—Average speeds of passenger cars observed during study periods on two-lane highways

Position of object on shoulder	Daytime								Nighttime			
	16-foot pavement		20-foot pavement		22-foot pavement		26-foot pavement		20-foot pavement		22-foot pavement	
	Near [1]	Far [2]	Near	Far	Near	Far	Near	Far	Near	Far	Near	Far
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Shoulder clear	47.1	42.9	45.2	43.8	48.7	48.9	45.9	46.1	43.1	41.1	------	------
Car at edge of pavement	45.0	43.9	42.6	44.2	47.0	48.0	45.6	45.2	40.8	41.6	------	------
Car 3 feet from edge of pavement	45.8	43.2	42.5	42.9	49.9	48.5	45.8	44.9	43.5	41.8	46.3	48.0
Car 6 feet from edge of pavement	46.5	44.4	41.8	43.0	49.2	47.6	42.9	42.1	43.7	45.0	44.9	43.9
Truck at edge of pavement	40.9	43.7	42.4	41.8	48.0	48.6	44.4	44.4	------	------	------	------
Truck 3 feet from edge of pavement	45.9	43.9	42.4	43.4	48.4	48.7	45.8	46.0	------	------	------	------
Truck 6 feet from edge of pavement	45.3	43.2	43.4	44.3	49.2	49.1	44.6	44.3	------	------	------	------
Barricade at edge of pavement	42.6	42.4	40.6	43.4	47.1	47.0	42.0	43.2	41.7	44.6	40.6	42.2
Barricade 3 feet from edge of pavement	44.4	43.3	40.7	42.1	48.0	44.2	44.0	42.0	39.2	39.9	42.6	43.9
Barricade 6 feet from edge of pavement	44.3	42.9	39.6	42.2	47.8	45.2	44.6	44.9	40.7	40.6	44.7	44.8

[1] Vehicles traveling in lane adjacent to occupied shoulder.
[2] Vehicles traveling in lane adjacent to clear shoulder.

Table 4.—Average speeds of commercial vehicles observed during study periods on two-lane highways

Position of object on shoulder	Daytime								Nighttime			
	16-foot pavement		20-foot pavement		22-foot pavement		26-foot pavement		20-foot pavement		22-foot pavement	
	Near [1]	Far [2]	Near	Far	Near	Far	Near	Far	Near	Far	Near	Far
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Shoulder clear	43.2	38.1	38.5	37.6	40.8	42.4	37.6	39.1	35.6	36.2	------	-------
Car at edge of pavement	40.8	37.5	37.6	37.2	38.6	42.9	41.7	39.0	35.2	34.6	------	------
Car 3 feet from edge of pavement	42.3	39.7	36.9	38.5	42.9	41.2	37.6	37.4	36.4	37.6	44.2	37.8
Car 6 feet from edge of pavement	42.4	38.7	34.4	38.5	40.3	42.0	37.8	37.4	37.0	37.7	40.3	41.0
Truck at edge of pavement	38.4	37.3	36.9	36.5	40.5	41.6	39.6	40.8	------	------	------	------
Truck 3 feet from edge of pavement	41.8	41.5	36.9	37.0	42.3	43.3	37.4	38.5	------	------	------	------
Truck 6 feet from edge of pavement	40.4	35.1	37.8	39.5	42.0	41.9	38.1	38.3	------	------	------	------
Barricade at edge of pavement	40.8	38.0	34.1	36.8	43.4	39.6	37.6	37.6	35.5	40.3	37.6	37.7
Barricade 3 feet from edge of pavement	42.3	39.0	35.4	39.1	40.7	41.9	38.0	36.8	31.2	37.2	38.5	37.5
Barricade 6 feet from edge of pavement	44.0	40.6	40.4	40.3	41.0	42.5	40.7	34.5	33.8	34.2	36.9	38.9

[1] Vehicles traveling in lane adjacent to occupied shoulder.
[2] Vehicles traveling in lane adjacent to clear shoulder.

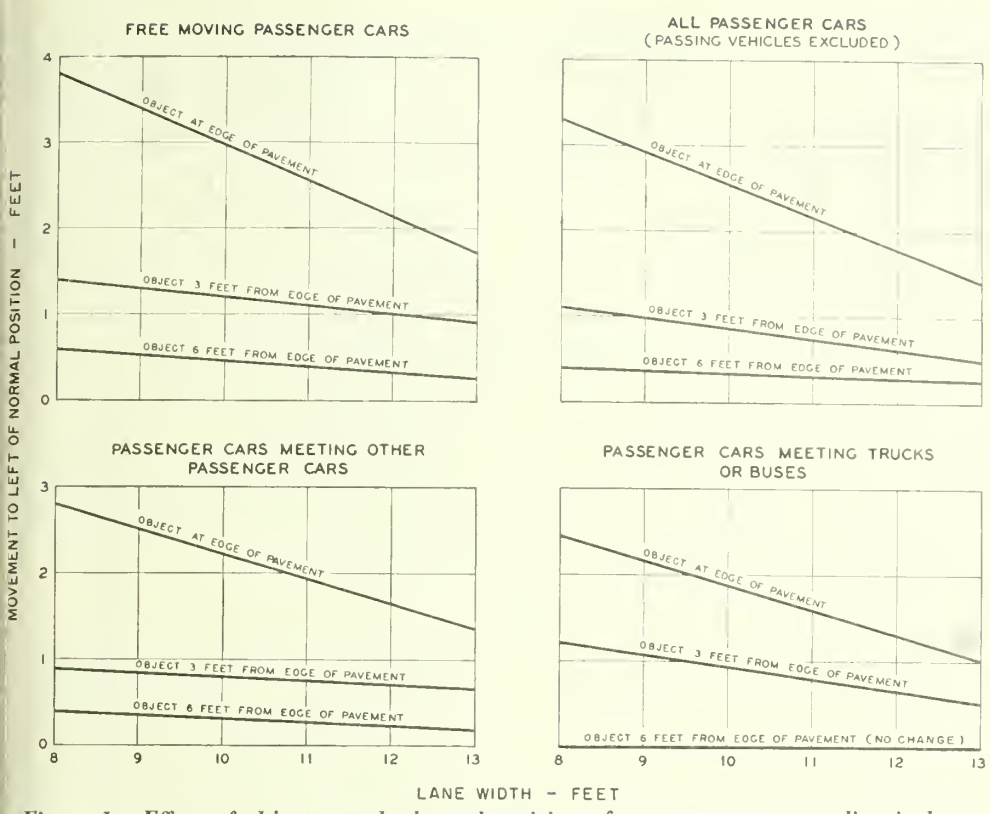

FREE MOVING PASSENGER CARS
ALL PASSENGER CARS
(PASSING VEHICLES EXCLUDED)
OBJECT AT EDGE OF PAVEMENT
OBJECT 3 FEET FROM EDGE OF PAVEMENT
OBJECT 6 FEET FROM EDGE OF PAVEMENT
PASSENGER CARS MEETING OTHER PASSENGER CARS
PASSENGER CARS MEETING TRUCKS OR BUSES
OBJECT 6 FEET FROM EDGE OF PAVEMENT (NO CHANGE)
MOVEMENT TO LEFT OF NORMAL POSITION - FEET
LANE WIDTH - FEET

Figure 1.—Effect of objects on the lateral position of passenger cars traveling in lane adjacent to occupied shoulder.

miles per hour slower than passenger cars. In addition to the speeds for normal conditions, tables 3 and 4 show average speeds while each of the objects was on the shoulder at various distances from the pavement edge.

On the four-lane divided highway during normal conditions, and at volumes below 2,000 vehicles per hour in one direction, passenger cars averaged about 50 miles per hour in the left lane and 43 miles per hour in the right lane. Commercial vehicles under similar conditions traveled about 2 miles per hour slower than passenger cars. The average speed on the four-lane highway when carrying volumes above 2,000 vehicles per hour was far more dependent on other conditions, which could not be held constant, than on the shoulder condition. For this reason, therefore, analysis of the effect of an object on the shoulder on vehicle speeds on the four-lane road was confined to volumes below 2,000 vehicles per hour.

Effect on Speeds

A parked vehicle or a barricade on the shoulder of the two-lane highways included in this study had a general tendency to reduce vehicle speeds. The reduction, however, was not consistently greater for one of the objects than for another. Also, the reduction was

Table 5.—Change in average speeds due to an object on one shoulder of a two-lane highway compared with normal speeds

Type of object on shoulder	Distance of object from pavement edge	Change in average speeds for—				
		16-foot pavement [1]	20-foot pavement	22-foot pavement	26-foot pavement	All pavement widths
Passenger Cars Traveling in Lane Adjacent to Clear Shoulder						
	Feet	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Car	0	1.0	0.4	−0.9	−0.9	−0.1
	3	.3	−.9	−.4	−1.2	−.6
	6	1.5	−.8	−1.3	−4.0	−1.2
Truck	0	.8	−2.0	−.3	−1.7	−.8
	3	1.0	−.4	−.2	−.1	.1
	6	.3	.5	.2	−1.8	−.2
Barricade	0	−.5	−.4	−1.9	−2.9	−1.4
	3	.4	−1.7	−4.7	−4.1	−2.5
	6	0	−1.6	−3.7	−1.2	−1.6
Average, by type of object:						
Car	-----	.9	−.4	−.9	−2.0	−.6
Truck	-----	.7	−.6	−.1	−1.2	−.3
Barricade	-----	0	−1.2	−3.4	−2.7	−1.8
Average, by distance of object:						
0 feet	-----	.4	−.7	−1.0	−1.8	−.8
3 feet	-----	.6	−1.0	−1.8	−1.8	−1.0
6 feet	-----	.6	−.6	−1.6	−2.3	−1.0
Average, all conditions	-----	.5	−.8	−1.5	−2.0	−.9
Commercial Vehicles Traveling in Lane Adjacent to Clear Shoulder						
Car	0	−0.6	−0.4	0.5	−0.1	−0.2
	3	1.6	.9	−1.2	−1.7	−.1
	6	.6	.9	−.4	−1.7	−.2
Truck	0	−.8	−1.1	−.8	1.7	−.3
	3	2.6	−.6	.9	−.6	.6
	6	−3.0	−1.9	−.5	−.8	−1.6
Barricade	0	−.1	−.8	−2.8	−1.5	−1.3
	3	.9	1.5	−.5	−2.3	−.1
	6	2.5	2.7	.1	−4.6	.2
Average, by type of object:						
Car	-----	.5	.5	−.4	−1.2	−.2
Truck	-----	−.4	−1.2	−.1	.1	−.4
Barricade	-----	1.1	1.1	−1.1	−2.8	−.4
Average, by distance of object:						
0 feet	-----	−.5	−.8	−1.0	0	−.6
3 feet	-----	1.7	.6	−.3	−1.5	.1
6 feet	-----	0	.6	−.3	−2.4	−.5
Average, all conditions	-----	.4	.1	−.5	−1.3	−.3

[1] 16 feet of portland cement concrete with 2-foot bituminous shoulders.

Table 6.—Change in average speeds due to an object on one shoulder of a two-lane highway compared with normal speeds

Type of object on shoulder	Distance of object from pavement edge	Change in average speeds for—				
		16-foot pavement [1]	20-foot pavement	22-foot pavement	26-foot pavement	All pavement widths
Passenger Cars Traveling in Lane Adjacent to Occupied Shoulder						
	Feet	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Car	0	−2.1	−2.6	−1.7	−0.3	−1.7
	3	−1.3	−2.7	1.2	−.1	−.7
	6	−.6	−3.4	.5	−3.0	−1.6
Truck	0	−6.2	−2.8	−.7	−1.5	−2.8
	3	−1.2	−2.8	−.3	−.1	−1.1
	6	−1.8	−1.8	.5	−1.3	−1.1
Barricade	0	−4.5	−4.6	−1.6	−3.9	−3.6
	3	−2.7	−4.5	−.7	−1.9	−2.4
	6	−2.8	−5.6	−.9	−1.3	−2.6
Average by type of object:						
Car	-----	−1.3	−2.9	0	−1.1	−1.3
Truck	-----	−3.1	−2.5	−.2	−1.0	−1.7
Barricade	-----	−3.3	−4.9	−1.1	−2.4	−2.9
Average, by distance of object:						
0 feet	-----	−4.3	−3.3	−1.3	−1.9	−2.7
3 feet	-----	−1.7	−3.3	.1	−.7	−1.4
6 feet	-----	−1.7	−3.6	0	−1.9	−1.8
Average, all conditions	-----	−2.6	−3.4	−.4	−1.5	−2.0
Commercial Vehicles Traveling in Lane Adjacent to Occupied Shoulder						
Car	0	−2.4	−0.9	−2.2	4.1	−0.4
	3	−.9	−1.6	2.1	0	−.1
	6	−.8	−4.1	−.5	.2	−1.3
Truck	0	−4.8	−1.6	−.3	2.0	−1.2
	3	−1.4	−1.6	1.5	−.2	−.4
	6	−2.8	−.7	1.2	.5	−.4
Barricade	0	−2.8	−4.4	2.6	0	−1.2
	3	−.9	−3.1	−.1	.4	−.9
	6	.8	1.9	.2	3.1	1.5
Average, by type of object:						
Car	-----	−1.4	−2.2	−.2	1.4	−.6
Truck	-----	−3.0	−1.3	.8	.8	−.7
Barricade	-----	−1.0	−1.9	.9	1.2	−.2
Average, by distance of object:						
0 feet	-----	−3.3	−2.3	0	2.0	−.9
3 feet	-----	−1.1	−2.1	1.2	.1	−.5
6 feet	-----	−.9	−1.0	.3	1.3	−.1
Average, all conditions	-----	−1.8	−1.8	.5	1.1	−.5

[1] 16 feet of portland cement concrete with 2-foot bituminous shoulders.

Table 7.—Change in average speeds due to an object on one shoulder of a two-lane highway compared with normal speeds

Type of object on shoulder	Distance of object from pavement edge	Change in average speeds for—				
		16-foot pavement [1]	20-foot pavement	22-foot pavement	26-foot pavement	All pavement widths
PASSENGER CARS IN LANE ADJACENT TO CLEAR SHOULDER AND MEETING OTHER VEHICLES						
	Feet	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Car	0	2.1	−0.3	−0.2	−1.5	0
	3	−2.3	−1.1	0	−4.4	−2.0
	6	1.4	−2.0	.5	−5.9	−1.5
Truck	0	1.7	−2.7	1.3	−3.6	−.8
	3	−.1	−1.9	−.5	−1.6	−1.0
	6	.4	−1.0	3.2	−4.3	−.4
Barricade	0	−.9	−1.5	.4	−5.5	−1.9
	3	.4	−4.3	−4.6	−5.7	−3.6
	6	−1.1	−2.0	−2.5	−5.3	−2.7
Average, by type of object:						
Car	-------	.4	−1.1	.1	−3.9	−1.2
Truck	-------	.7	−1.9	1.3	−3.2	−.7
Barricade	-------	−.5	−2.6	−2.2	−5.5	−2.7
Average, by distance of object:						
0 feet	-------	1.0	−1.5	.5	−3.5	−.9
3 feet	-------	−.7	−2.4	−1.7	−3.9	−2.2
6 feet	-------	.2	−1.7	−.4	−5.2	−1.5
Average, all conditions	-------	.2	−1.9	−.3	−4.2	−1.5
PASSENGER CARS IN LANE ADJACENT TO OCCUPIED SHOULDER AND MEETING OTHER VEHICLES						
Car	0	−5.6	−2.6	−3.3	−2.0	−3.4
	3	.1	−1.5	1.5	0	0
	6	.6	−3.0	−1.9	−4.3	−2.2
Truck	0	−5.5	−1.5	−2.0	−1.9	−2.7
	3	−1.8	−3.5	−1.6	−1.4	−2.1
	6	−1.3	−3.7	−3.2	−1.1	−2.3
Barricade	0	−6.8	−2.9	−2.0	−5.1	−4.2
	3	−2.5	−3.2	−2.5	−1.4	−2.4
	6	−.1	−3.0	−2.0	−1.8	−1.7
Average, by type of object:						
Car	-------	−1.6	−2.4	−1.2	−2.1	−1.9
Truck	-------	−2.9	−2.9	−2.3	−1.5	−2.4
Barricade	-------	−3.1	−3.0	−2.2	−2.8	−2.8
Average, by distance of object:						
0 feet	-------	−6.0	−2.3	−2.4	−3.0	−3.4
3 feet	-------	−1.4	−2.7	−.9	−.9	−1.5
6 feet	-------	−.3	−3.2	−2.4	−2.4	−2.1
Average, all conditions	-------	−2.5	−2.8	−1.7	−2.1	−2.3

[1] 16 feet of portland cement concrete with 2-foot bituminous shoulders.

not consistently greater when the object on the shoulder was placed at the edge of the pavement than when located 3 or 6 feet from the edge, although there was some tendency in this direction.

The results are not surprising because a number of other studies have shown that speeds are not reduced as much as one would expect by unusual conditions. A study of speeds at short, narrow, two-lane bridges,[2] for example, showed that drivers did not reduce their speeds appreciably as they approached the bridges although the drivers did make a considerable change in the lateral position of their vehicles—shying away from the bridge railings or parapet walls. The changes in speed caused by placing the three types of objects on the shoulder at various distances from the pavement edge are shown in tables 5–8. The following statements are based on a study of the data shown by these tables:

1. There is only a slight tendency for passenger car drivers to reduce their speeds when traveling in the lane adjacent to the unoccupied shoulder. On an average, the reduction in speed was less than 1 mile per hour. The surface width, type of object, or its location on the other shoulder made little difference.

2. The average passenger car driver traveling in the lane adjacent to the occupied shoulder reduced his speed an average of 3 miles per hour on two-lane pavements 16 and 20 feet wide, and an average of 1 mile per hour on pavements 22 and 26 feet wide. There was a somewhat greater tendency under these conditions for the drivers to reduce their speeds with a barricade on the shoulder than with a truck or passenger car parked on the shoulder. The distance of the object from the pavement edge, however, made no consistent difference.

3. Truck drivers, regardless of the lane in which they were traveling, were influenced by the shoulder condition even less than passenger car drivers.

4. The average passenger car driver, meeting another vehicle traveling in the opposite direction at the same place on the highway as the object was located on the shoulder, reduced his speed 2.3 miles per hour if in the lane adjacent to the occupied shoulder, and 1.5 miles per hour if in the other lane. Under these conditions the type of object, its location on the shoulder, and the pavement width made little difference.

5. On the four-lane highway there was no consistent tendency for drivers in either lane under any of the study conditions to change their speeds with respect to those under normal conditions.

Effect on Lateral Position

Numerous studies have shown that the effect on driver behavior when any geometric feature of the cross section of a highway is changed will generally be reflected in a change in the lateral position of the vehicles. For this purpose, lateral positions have usually been a more definite criterion than vehicle speeds or any other characteristic of driver behavior that can be measured.

Two-lane highways

Neither the parked car, truck, nor barricade had any effect on the lateral position of vehicles traveling in a traffic lane not adjacent to the shoulder on which these objects were located. The differences between the lateral positions of the vehicles in this lane under normal conditions and when the objects were located on the shoulder are shown in table 9. In nearly all cases the values in this table, representing the change in lateral positions, are 0.3 of a foot or less, the precision to which the lateral positions were measured. There is no consistent tendency for the change in

Table 8.—Change in average speeds due to an object on the shoulder of a four-lane divided highway compared with normal speeds [1]

Position of object on shoulder	Change in average speeds in—			
	Left lane		Right lane	
	Passenger cars	Commercial vehicles	Passenger cars	Commercial vehicles
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*
Car at pavement edge	0.1	1.9	−0.7	0
Car 3 feet from pavement edge	.4	−.9	−1.1	−.4
Car 6 feet from pavement edge	.7	1.3	−1.1	−1.3
Truck at pavement edge	−.5	1.5	−.4	−1.3
Truck 3 feet from pavement edge	−.6	.7	3.0	3.0
Truck 6 feet from pavement edge	−3.5	−2.7	.6	1.0
Barricade at pavement edge	−.9	−1.1	0	−.8
Barricade 3 feet from pavement edge	−1.0	−1.1	1.6	.5
Barricade 6 feet from pavement edge	−2.0	−3.4	.4	1.2
Average, by type of object:				
Car	.4	.8	−1.0	−.6
Truck	−1.5	−.2	1.1	.9
Barricade	−1.3	−1.9	.7	.3
Average, by distance of object:				
0 feet	−.4	.8	−.4	−.7
3 feet	−.4	−.4	1.2	1.0
6 feet	−1.6	−1.6	0	.3
Average, all conditions	−.8	−.4	.2	.2

[1] Traffic volume less than 2,000 vehicles per hour in the two lanes.

[2] *Influence of bridge widths on transverse positions of vehicles*, by W. P. Walker, proceedings of the Highway Research Board, 1941, vol. 21, p. 361.

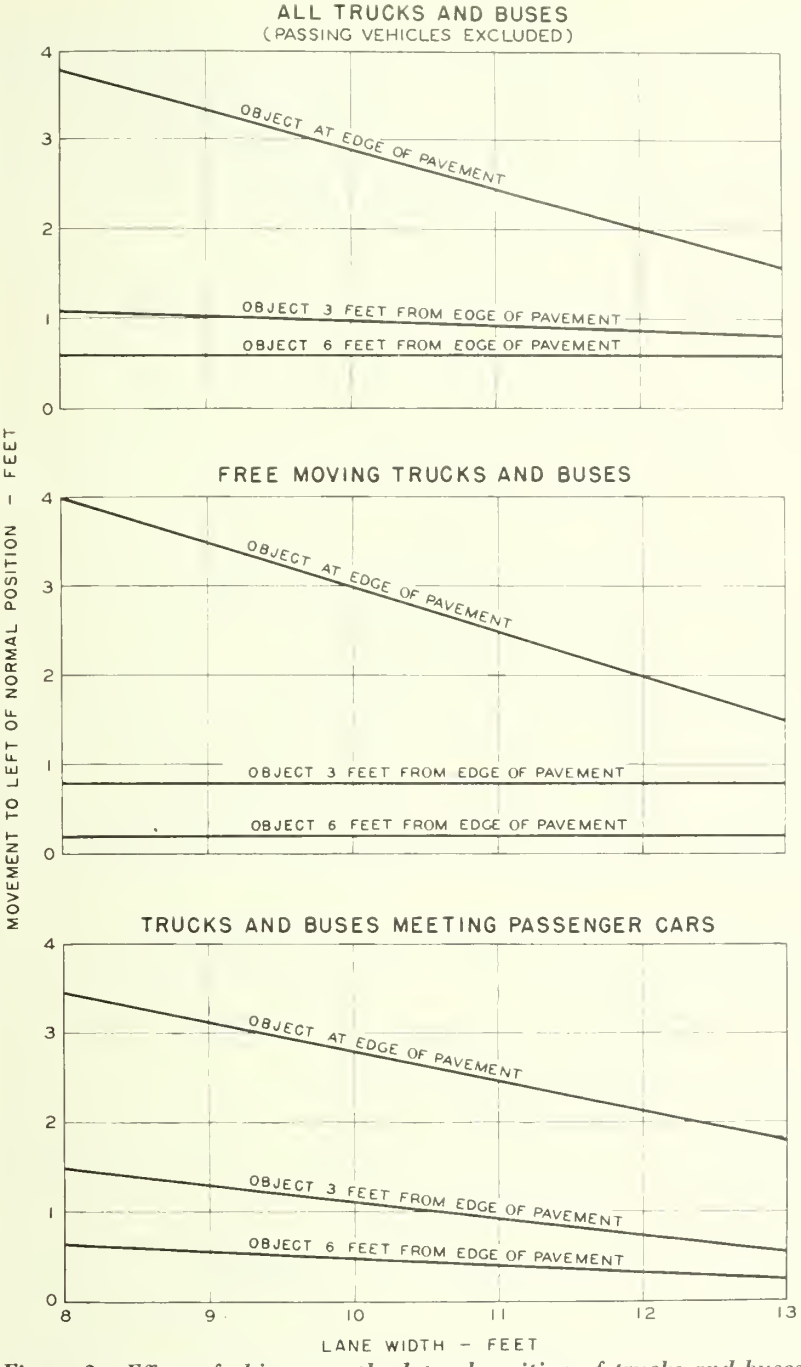


Figure 2.—Effect of objects on the lateral position of trucks and buses traveling in lane adjacent to occupied shoulder.

position to be larger or smaller for the narrower surfaces than for the wider surfaces or for one type or location of object than for another type or location of object.

The objects on the shoulder did, however, have a very definite effect on the lateral positions of vehicles traveling in the lane adjacent to the shoulder on which the objects were placed, as shown in table 10. This table also shows that the changes in the lateral positions because of an object on the shoulder were greater for the narrow surfaces than for the wider ones, and that they were greater when the objects on the shoulder were closer to the pavement edge.

The type of object, however, had no consistent effect on the results. An analysis of the clearances between vehicles when meeting on the two-lane highways also showed that there was relatively little difference between the results obtained with either the car, truck, or barricade on the shoulder. For this reason, the figures for two-lane roads showing the effect that an object parked on a shoulder has on the lateral position of a vehicle traveling in the adjacent lane are applicable whether the parked object is a car, truck, or barricade.

The change in the lateral position of passenger cars caused by a vehicle or barricade being parked on the shoulder adjacent to the traffic lane in which the passenger car was traveling is shown in figure 1. Similar information is shown for commercial vehicles in figure 2. The change in the lateral positions in these and all subsequent figures is shown in terms of the movement to the left of the position that the vehicle occupied under normal conditions. In other words, it is the distance that the average vehicle, of a certain group of vehicles, shifted its position toward the centerline of the highway and away from the occupied shoulder.

The drivers of free-moving passenger cars or drivers not influenced by the presence of other traffic on the highway made the greatest shift toward the centerline and away from their normal position because of objects placed on the right shoulder. On the other hand, drivers of passenger cars meeting a truck or bus at a point adjacent to the vehicle or barricade placed on the shoulder made the smallest change from their normal paths. Under this latter condition, however, the movement to the left was 1 foot when the object was placed at the edge of a 13-foot lane and 2.5 feet when the object was at the edge of an 8-foot lane. The only drivers not changing their lateral positions were those who met commercial vehicles when the objects on the shoulder were 6 feet from the pavement edge.

The objects on the shoulders generally caused trucks to veer farther than passenger cars from their normal paths (figs. 1–2). There were some exceptions, however, such as the effect on free-moving vehicles when the objects were either 3 or 6 feet from the pavement. Under these conditions, the passenger cars veered away from the objects more than trucks.

Clearance Between Vehicles

Two-lane roads carrying two-directional traffic should be of sufficient width to provide adequate clearance between the bodies of meeting vehicles. The results of previous studies have shown that 3 feet is the minimum desirable lateral clearance for this condition and that clearances of less than 3 feet occur infrequently on two-lane roads with 12-foot traffic lanes and adequate shoulders. This study shows, however, that when an object such as a car, truck, or barricade was placed on one shoulder at the edge of a two-lane road with a 24-foot surface, 30 percent of the passenger cars had a clearance of 3 feet or less when meeting other passenger cars (fig. 3). On surfaces with 11-foot traffic lanes the corresponding figure was 46 percent. Surfaces with 8-foot traffic lanes are obviously too narrow for modern vehicles and speeds, since 56 percent of the meetings of passenger cars took place with clearances of 3 feet or less although the average clearance exceeded 3 feet under normal conditions when the shoulders were clear. Placing a car, truck, or barricade on one of the shoulders increased the percentage from 56 to 94 and decreased the average clearance between vehicles from 3.2 to 1.3 feet (fig. 3).

Figure 4, which is similar to figure 3, shows clearances between bodies of passenger cars and trucks as they met. In figure 4 the average clearances are less and the percentage of vehicles with clearances of 3 feet or less are greater than in figure 3. This is to be expected because trucks are wider than passenger cars.

Crossing of Centerline

The average position of both passenger cars and trucks under normal conditions is to the

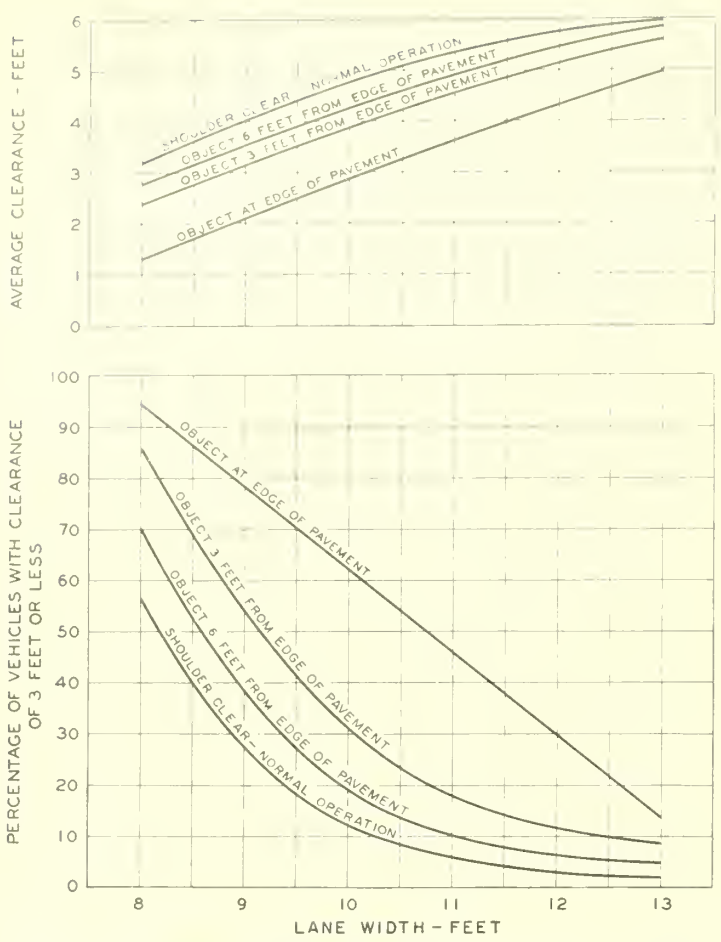

AVERAGE CLEARANCE - FEET
SHOULDER CLEAR - NORMAL OPERATION
OBJECT 6 FEET FROM EDGE OF PAVEMENT
OBJECT 3 FEET FROM EDGE OF PAVEMENT
OBJECT AT EDGE OF PAVEMENT
PERCENTAGE OF VEHICLES WITH CLEARANCE OF 3 FEET OR LESS
OBJECT AT EDGE OF PAVEMENT
OBJECT 3 FEET FROM EDGE OF PAVEMENT
OBJECT 6 FEET FROM EDGE OF PAVEMENT
SHOULDER CLEAR - NORMAL OPERATION
LANE WIDTH - FEET

Figure 3.—Clearances between bodies of passenger cars when meeting.

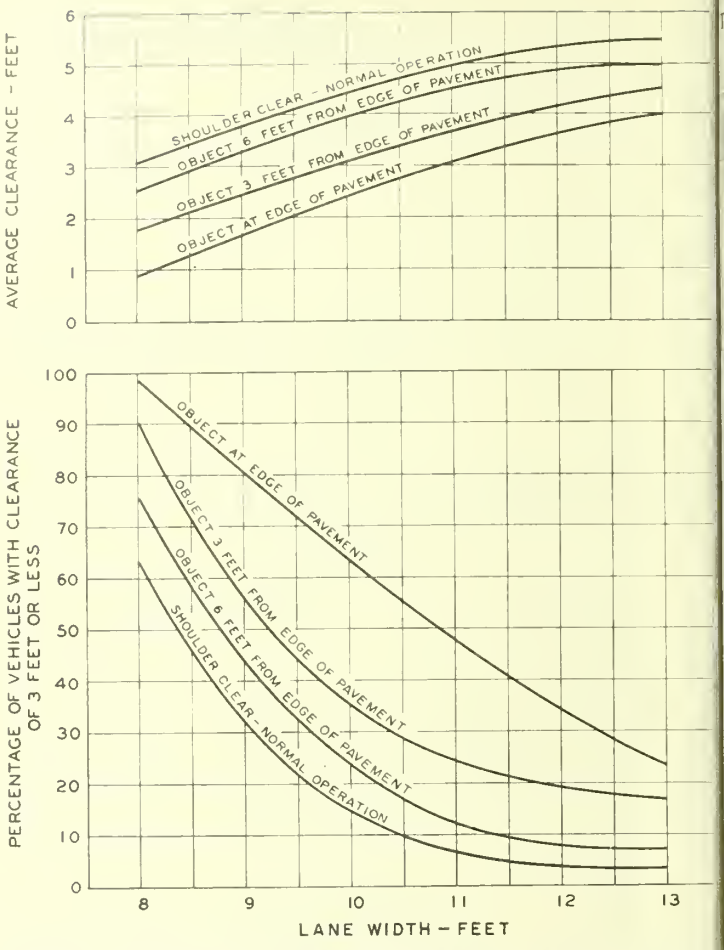

AVERAGE CLEARANCE - FEET
SHOULDER CLEAR - NORMAL OPERATION
OBJECT 6 FEET FROM EDGE OF PAVEMENT
OBJECT 3 FEET FROM EDGE OF PAVEMENT
OBJECT AT EDGE OF PAVEMENT
PERCENTAGE OF VEHICLES WITH CLEARANCE OF 3 FEET OR LESS
OBJECT AT EDGE OF PAVEMENT
OBJECT 3 FEET FROM EDGE OF PAVEMENT
OBJECT 6 FEET FROM EDGE OF PAVEMENT
SHOULDER CLEAR - NORMAL OPERATION
LANE WIDTH - FEET

Figure 4.—Clearances between bodies when passenger cars mee trucks or buses.

Table 9.—Change in lateral position of vehicles due to an object on one shoulder of a two-lane highway compared with normal conditions

Type of object on shoulder	Distance of object from pavement edge	Change in lateral position for—			
		16-foot pavement	20-foot pavement	22-foot pavement	26-foot pavement
PASSENGER CARS TRAVELING IN LANE ADJACENT TO CLEAR SHOULDER					
	Feet	*Feet*	*Feet*	*Feet*	*Feet*
Car	0	0.1	−0.3	−0.1	−0.2
	3	.1	−.5	−.1	0
	6	.1	−.6	−.1	0
Truck	0	.2	−.2	.4	.1
	3	.1	−.3	.1	−.1
	6	.2	−.3	.1	0
Barricade	0	.1	−.2	.2	.3
	3	−.2	−.3	.1	.3
	6	−.1	−.4	.1	.2
Average, by distance of object:					
0 feet	-------	.1	−.2	−.2	−.1
3 feet	-------	0	−.4	0	.1
6 feet	-------	.1	−.4	0	.1
COMMERCIAL VEHICLES TRAVELING IN LANE ADJACENT TO CLEAR SHOULDER					
Car	0	0.6	0.2	−0.3	−0.1
	3	.3	−.1	−.2	−.2
	6	.4	−.2	−.3	−.1
Truck	0	.4	0	.2	.3
	3	.1	−.1	−.1	0
	6	.1	−.2	0	.1
Barricade	0	.4	−.1	.1	.4
	3	0	−.4	−.2	0
	6	−.1	−.2	−.1	.1
Average, by distance of object:					
0 feet	-------	.5	0	0	.3
3 feet	-------	.1	−.2	−.2	−.1
6 feet	-------	.1	−.2	−.1	0

Table 10.—Change in lateral position of vehicles due to an objec on one shoulder of a two-lane highway compared with norma conditions

Type of object on shoulder	Distance of object from pavement edge	Change in lateral position for—			
		16-foot pavement	20-foot pavement	22-foot pavement	26-foot pavement
PASSENGER CARS TRAVELING IN LANE ADJACENT TO OCCUPIED SHOULDER					
	Feet	*Feet*	*Feet*	*Feet*	*Feet*
Car	0	3.0	2.1	2.6	1.0
	3	1.3	.4	1.0	.5
	6	.6	.3	.5	.3
Truck	0	3.7	2.6	2.8	1.7
	3	1.3	1.1	1.5	.8
	6	.6	.1	.7	.5
Barricade	0	3.4	2.1	2.0	1.6
	3	.9	.4	.9	.6
	6	.1	.2	.2	0
Average, by distance of object:					
0 feet	-------	3.4	2.3	2.5	1.4
3 feet	-------	1.2	.6	1.1	.6
6 feet	-------	.4	.2	.5	.3
COMMERCIAL VEHICLES TRAVELING IN LANE ADJACENT TO OCCUPIED SHOULDER					
Car	0	3.4	2.4	2.6	1.2
	3	.8	.7	1.0	.7
	6	.4	.3	.5	.3
Truck	0	3.9	3.0	3.0	1.9
	3	1.4	1.2	1.6	.9
	6	.6	.1	.7	.8
Barricade	0	3.8	2.2	2.3	1.8
	3	1.1	.5	.9	.8
	6	.5	.3	.7	.4
Average, by distance of object:					
0 feet	-------	3.7	2.5	2.6	1.6
3 feet	-------	1.1	.8	1.2	.8
6 feet	-------	.5	.2	.6	.5

Table 11.—Change in lateral position of vehicles due to an object on one shoulder of a four-lane highway compared with normal conditions

Type of object on shoulder	Distance of object from pavement edge	Change in lateral position for traffic volumes of—						
		1,000 v. p. h.	1,500 v. p. h.	2,000 v. p. h.	2,500 v. p. h.	3,000 v. p. h.	3,500 v. p. h.	All volumes
Passenger Cars Traveling in Left Lane								
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Car	0	0.1	0.1	0.1	0.1	0	0	0.1
	3	0	0	.1	.1	−0.1	−0.2	−.1
	6	.2	.1	.1	.1	0	0	.1
Truck	0	.1	.1	.1	0	0	0	.1
	3	.1	.1	.1	.1	.1	.1	.1
	6	−.1	−.1	−.1	−.1	−.2	−.2	−.1
Barricade	0	.2	.1	0	0	−.1	−.2	0
	3	−.1	−.1	−.1	−.1	−.1	−.2	−.1
	6	0	0	−.1	−.1	−.1	−.1	−.1
Average, by distance of object:								
0 feet	-----	.2	.1	.1	0	0	−.1	0
3 feet	-----	0	0	0	0	−.1	−.1	0
6 feet	-----	0	0	0	0	−.1	−.1	0
Commercial Vehicles Traveling in Left Lane [1]								
Car	0	---	−0.2	0.4	---	---	---	0.1
	3	---	0	.5	---	---	---	.2
	6	---	.3	.4	---	---	---	.3
Truck	0	---	.2	−.1	---	---	---	.1
	3	---	.1	.4	---	---	---	.2
	6	---	.2	.3	---	---	---	.2
Barricade	0	---	−.8	0	---	---	---	−.4
	3	---	.2	0	---	---	---	.1
	6	---	.3	.5	---	---	---	.4
Average, by distance of object:								
0 feet	-----	---	−.2	.1	---	---	---	0
3 feet	-----	---	.1	.3	---	---	---	.2
6 feet	-----	---	.2	.4	---	---	---	.3

[1] Traffic volumes are grouped into two classes: below 2,000 v. p. h. and 2,000 v. p. h. and over.

Table 12.—Change in lateral position of vehicles due to an object on one shoulder of a four-lane highway compared with normal conditions

Type of object on shoulder	Distance of object from pavement edge	Change in lateral position for traffic volumes of—						
		1,000 v. p. h.	1,500 v. p. h.	2,000 v. p. h.	2,500 v. p. h.	3,000 v. p. h.	3,500 v. p. h.	All volumes
Passenger Cars Traveling in Right Lane								
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Car	0	0.9	0.9	0.9	0.9	0.9	0.8	0.9
	3	.4	.4	.4	.4	.4	.3	.4
	6	.2	.2	.2	.2	.2	.1	.2
Truck	0	1.1	1.1	1.0	1.0	.9	.9	1.0
	3	.7	.6	.4	.3	.2	.1	.4
	6	0	0	.1	.1	.1	.2	.1
Barricade	0	.4	.4	.5	.5	.5	.5	.5
	3	.2	.2	.2	.2	.2	.2	.2
	6	−.1	−.1	0	0	0	0	0
Average, by distance of object:								
0 feet	-----	.8	.8	.8	.8	.8	.7	.8
3 feet	-----	.4	.4	.3	.3	.3	.2	.3
6 feet	-----	0	0	.1	.1	.1	.1	.1
Commercial Vehicles Traveling in Right Lane								
Car	0	1.0	1.0	0.9	0.9	0.8	0.8	0.9
	3	.1	.1	.1	.1	.1	.1	.1
	6	.2	.1	0	0	−.1	−.2	0
Truck	0	1.1	1.1	1.0	.9	.8	.7	.9
	3	.6	.5	.3	.2	.1	0	.3
	6	0	0	.1	.1	.2	.2	.1
Barricade	0	.4	.4	.4	.4	.4	.4	.4
	3	0	0	0	−.1	−.1	−.1	−.1
	6	−.1	−.1	−.1	−.1	−.1	−.1	−.1
Average, by distance of object:								
0 feet	-----	.8	.8	.4	.7	.7	.6	.7
3 feet	-----	.2	.2	.1	.1	0	0	.1
6 feet	-----	0	0	0	0	0	0	0

right of the centerline of a two-lane two-directional pavement even though it is only 16 feet wide. Figures 5–10 show that when a car, truck, or barricade was placed on one shoulder next to the edge of a 16- or 20-foot pavement, vehicles which normally traveled entirely in the lane adjacent to the occupied shoulder, encroached on the lane used by oncoming traffic. This was the case even in the face of oncoming traffic on a 16-foot surface with the result that oncoming traffic was forced to use the other shoulder (fig. 6).

When the objects were 6 feet from the edge of the pavement, regardless of the pavement width, they caused little change in the lateral positions of the vehicles as shown in figures 5–10. As an object on the shoulder was brought closer than 6 feet, there was an increasing effect on the lateral positions. This is true for all pavement widths but applies to a greater degree on the narrow two-lane pavements. (Figures presenting information on the 26-foot pavement were excluded from this article, because changes in lateral positions resulting from placement of objects on the shoulder were not too significant.) The effect on lateral positions is further illustrated by the chart at the right in figure 11 which combines the information in figures 1 and 2 in a somewhat different form to represent average traffic conditions during the study periods on the two-lane roads. Average traffic consisted of 10 percent commercial vehicles during volumes of 200 to 400 vehicles per hour.

The chart at the right in figure 11 shows the change in lateral positions which the objects caused. When an object was 6 or more feet from the pavement edge, the distance between a vehicle in the traffic stream and the pavement edge was increased by 0.4 foot when the traffic lanes were 8 feet wide and about 0.25 foot when the lanes were 13 feet wide. These measurements correspond to only 5 and 2 percent, respectively, of the lane widths, as shown by the chart at the left in figure 11. From these figures and from the shape of the curves in figure 11, it can be concluded that, for all practical purposes, objects 6 or more feet from the edge of the pavement of a two-lane road of any width have an insignificant effect on driver behavior. The effect of an object at the edge of a two-lane surface of any width is very significant and especially so for the narrower surfaces. Regardless of the surface width, however, an object 2 feet from the edge has only half the effect of an object at the edge of the surface.

Effect on Lateral Positions

Four-lane highways

The traffic volumes on the four-lane facility included in this study varied from 1,000 to over 3,400 vehicles per hour in one direction of travel. It is interesting to see how the normal pattern of vehicle placement on this highway varied with traffic volumes. This is shown in figures 12 and 13. As the volume increased, figure 12 shows that passenger cars in the right lane, on an average, traveled closer to the shoulder and those in the left lane traveled closer to the median. Furthermore, the average lateral position and hourly volume are linearly related. Similar results were obtained for commercial vehicles (fig. 13).

In order to eliminate the effect of volume and to obtain only the effect of objects on the shoulder on vehicle placement, comparisons of placement values were made at the same hourly volume. For this purpose it was necessary to determine graphically the average placements against the recorded volumes. The desired data were thus obtained for each 500-vehicle-per-hour increment within the range of observed volumes.

Vehicles in left lane

Table 11 shows the effect that the objects placed on the shoulder had on the lateral position of vehicles traveling in the left lane. The figures in the table are the differences between the lateral positions during normal conditions and when an object was on the shoulder. It can readily be seen that neither the type of object nor the distance that the object was from the pavement had any significant effect on lateral positions of passenger cars traveling in the left lane. Very few commercial vehicles traveled in the left lane. A detailed analysis of the effect of objects on these vehicles, therefore, could not be made. The limited data, however, do indicate that trucks in the left lane of a four-lane highway were also unaffected by objects on the shoulder.

Vehicles in right lane

Objects on the shoulder do affect the lateral positions of vehicles traveling in the right lane of a four-lane divided highway. The differences between the lateral positions of vehicles during normal conditions and when an object was on the shoulder are shown in table 12. A truck parked at the edge of the pavement or 3 feet away had a slightly greater effect on traffic than a parked car when the total hourly volume was 1,500 or less. Generally,

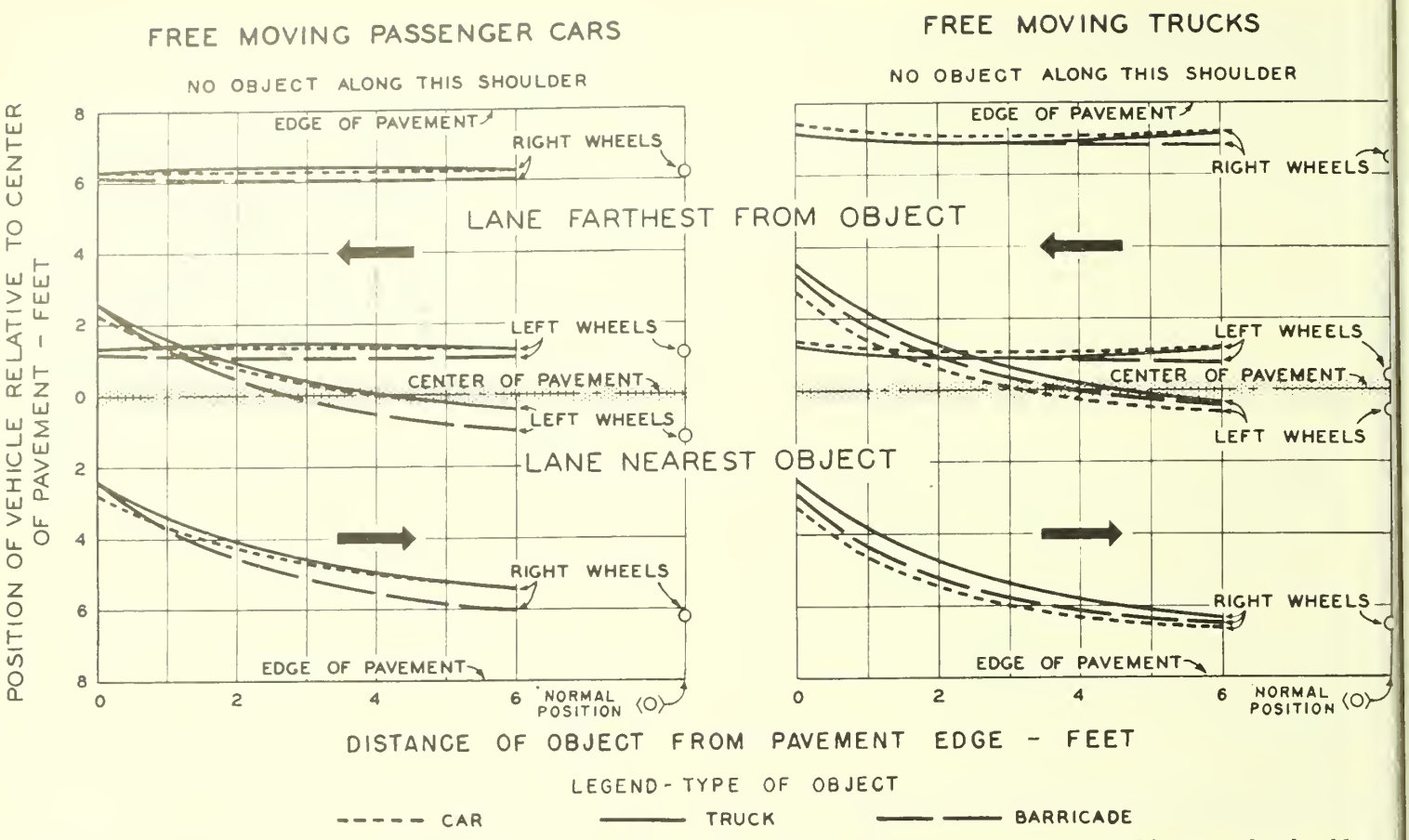


Figure 5.—Average position of free-moving vehicles traveling on a 16-foot concrete pavement with various objects on the shoulder.

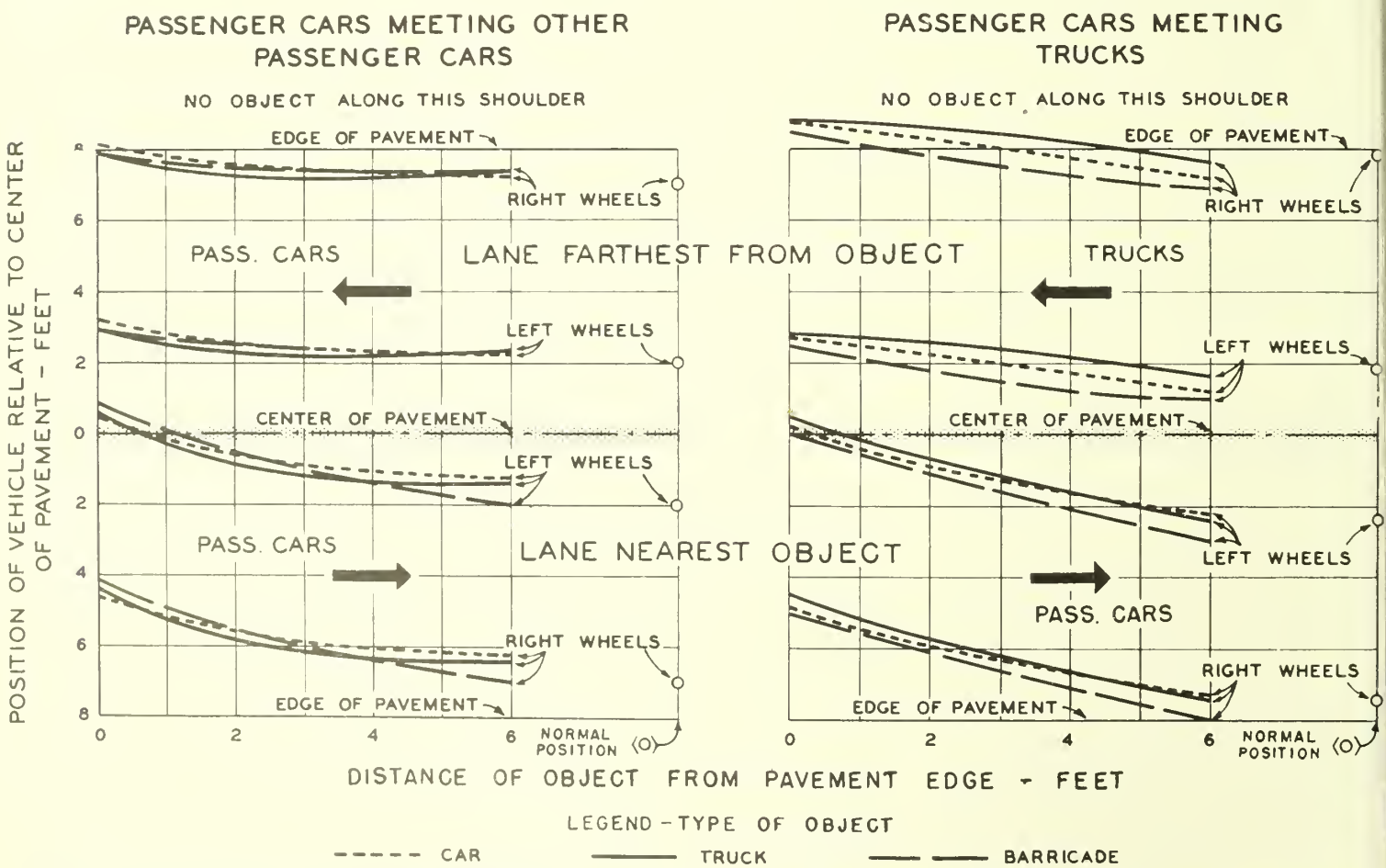


Figure 6.—Average position of vehicles meeting other vehicles on a 16-foot concrete pavement with various objects on the shoulder.

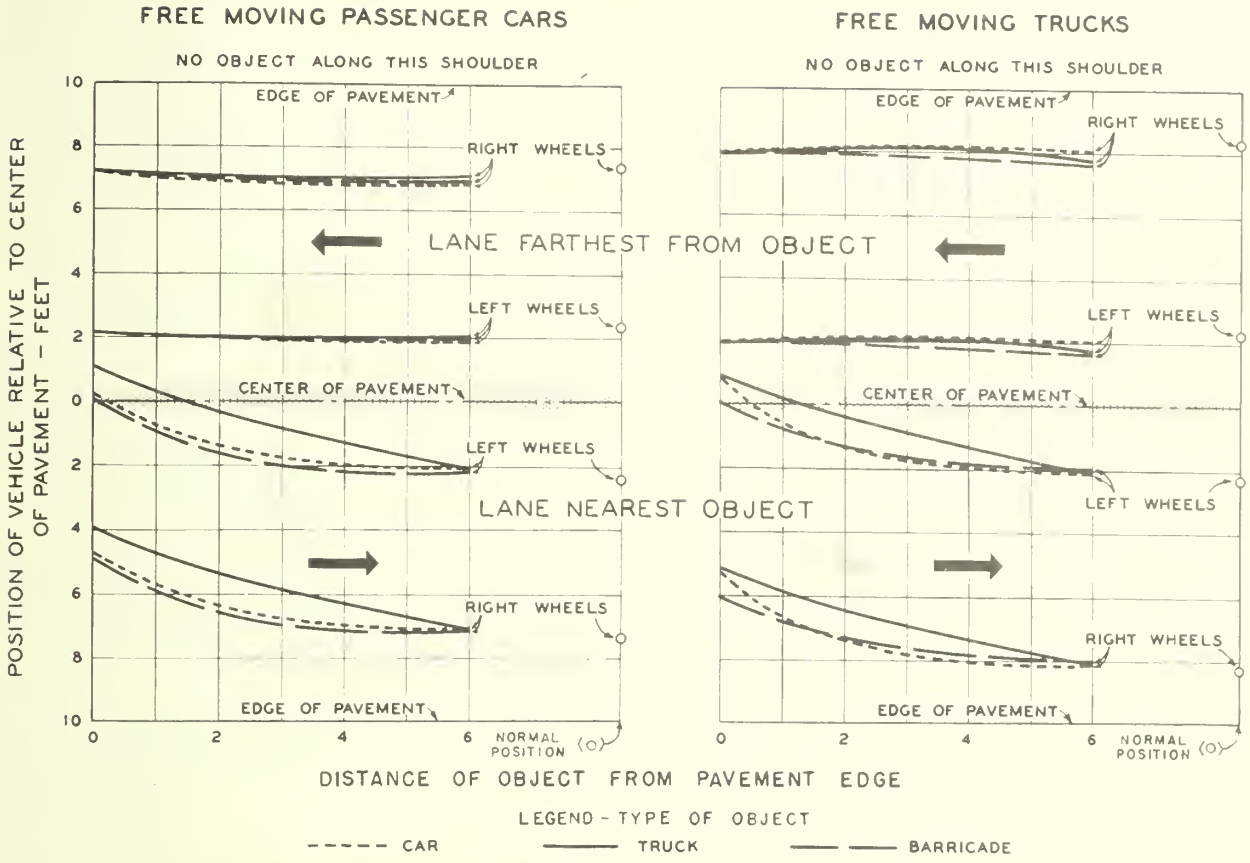


Figure 7.—Average position of free-moving vehicles traveling on a 20-foot concrete pavement with various objects on the shoulder.

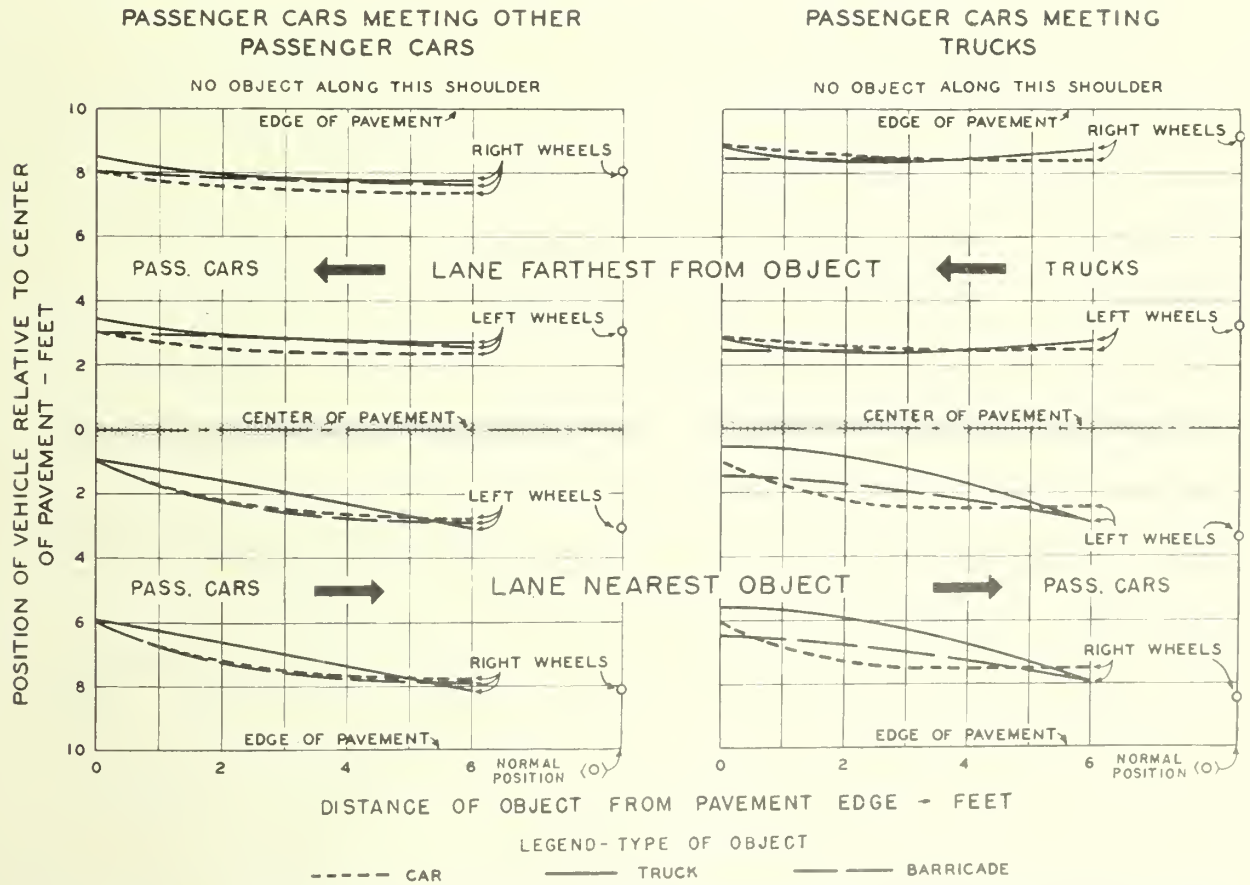


Figure 8.—Average position of vehicles meeting other vehicles on a 20-foot concrete pavement with various objects on the shoulder.

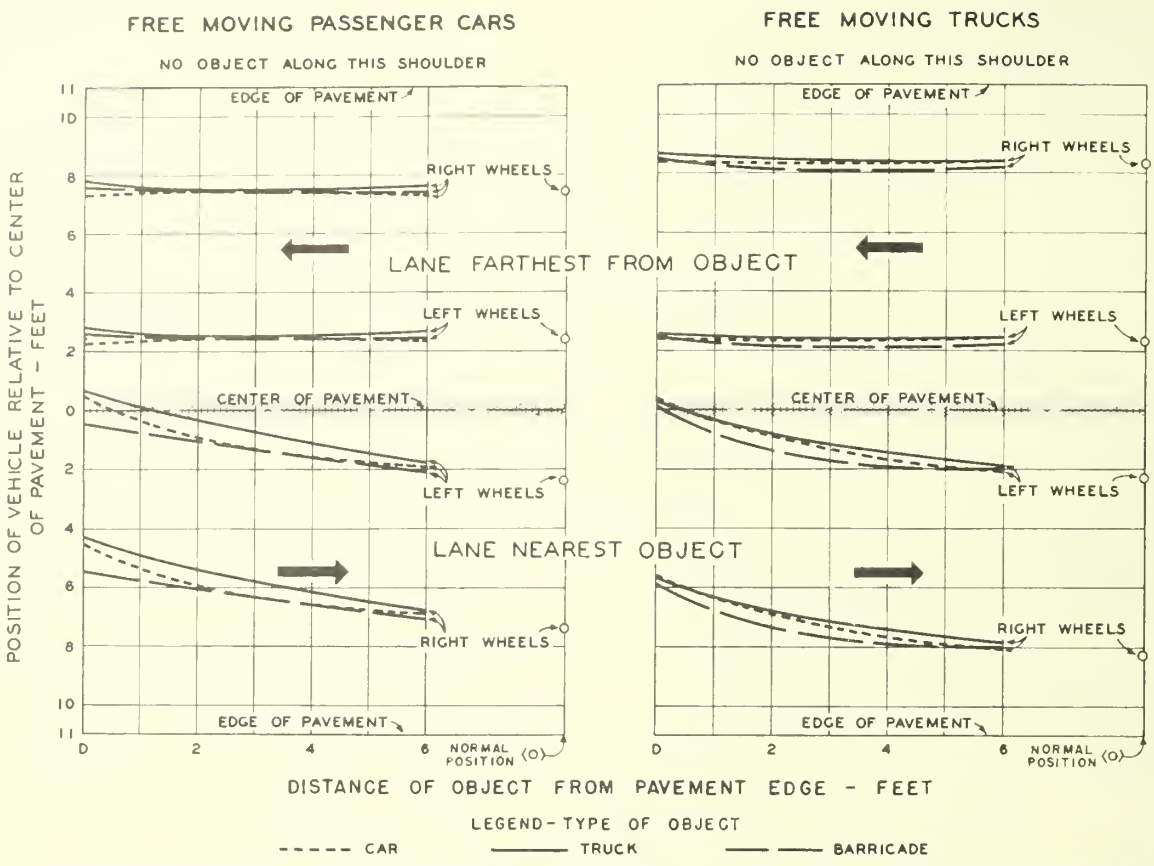


Figure 9.—Average position of free-moving vehicles traveling on a 22-foot concrete pavement with various objects on the shoulder.

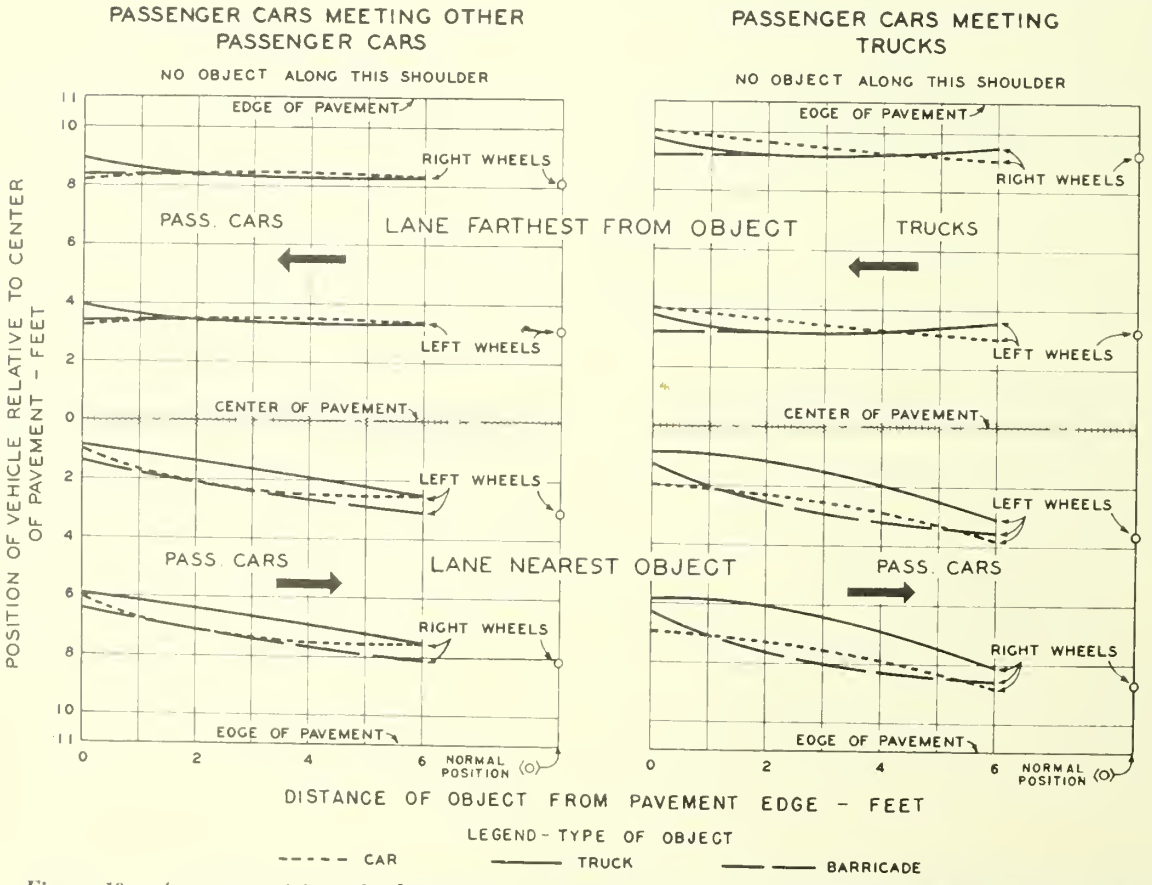


Figure 10.—Average position of vehicles meeting other vehicles on a 22-foot concrete pavement with various objects on the shoulder.

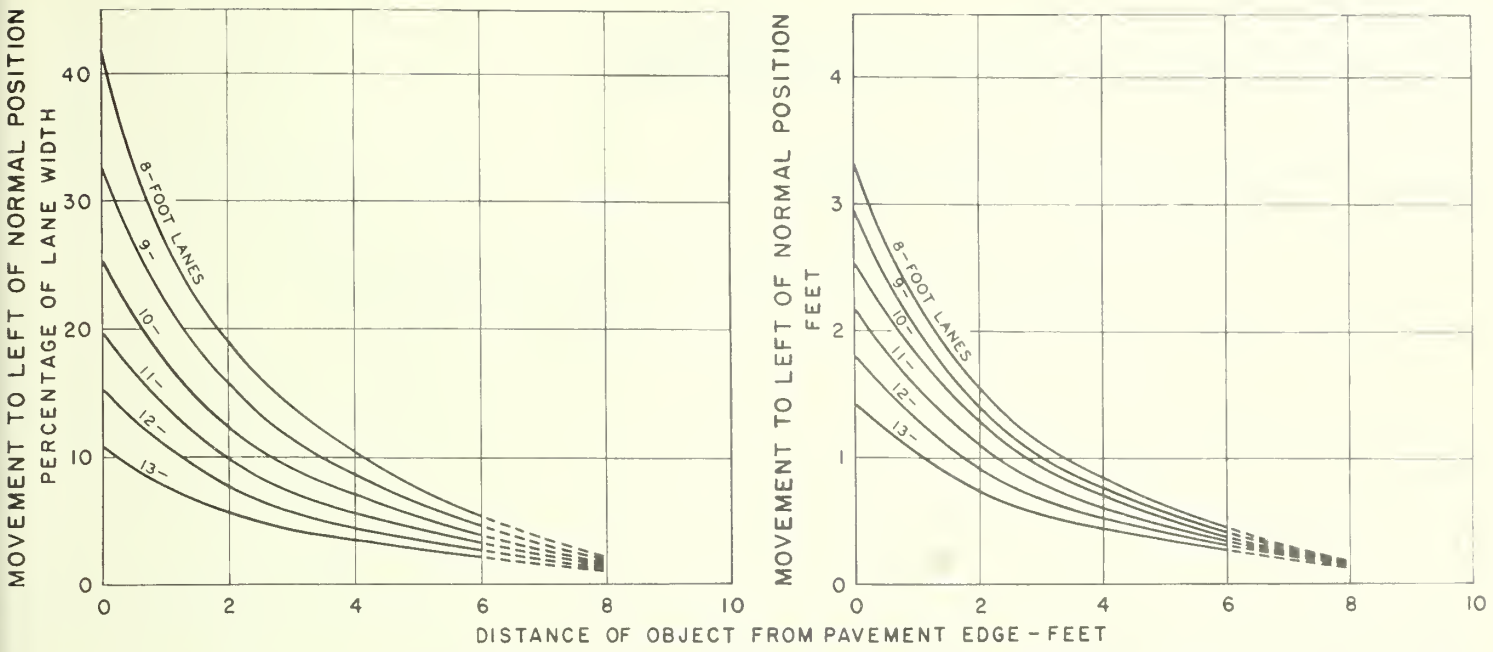


Figure 11.—Lateral movement of traffic, according to width of lanes, for various distances object was placed from pavement edge.

however, the effect of the truck was about the same as that of the car. A barricade placed on the shoulder had considerably less effect both on passenger cars and commercial vehicles than a parked car or truck under all traffic volume conditions.

When the total hourly volume in both traffic lanes was 1,000 vehicles per hour, passenger cars in the right lane traveled about a foot farther from the edge of the pavement when a truck or a car was parked at the pavement edge than when no object was on the shoulder. As the traffic volume increased, the effect of the parked truck or car, on lateral positions of vehicles decreased slightly.

A barricade at the edge of the surface had only about half the effect of a parked truck or car in causing passenger cars in the right lane to veer away from the shoulder. Objects on the shoulder 3 feet from the pavement edge had only half the effect as the same objects placed at the pavement edge. Any of the three objects placed 6 feet from the pavement edge had almost a negligible effect on passenger cars traveling in the right lane.

During low traffic volumes, commercial

(Continued on page 176)

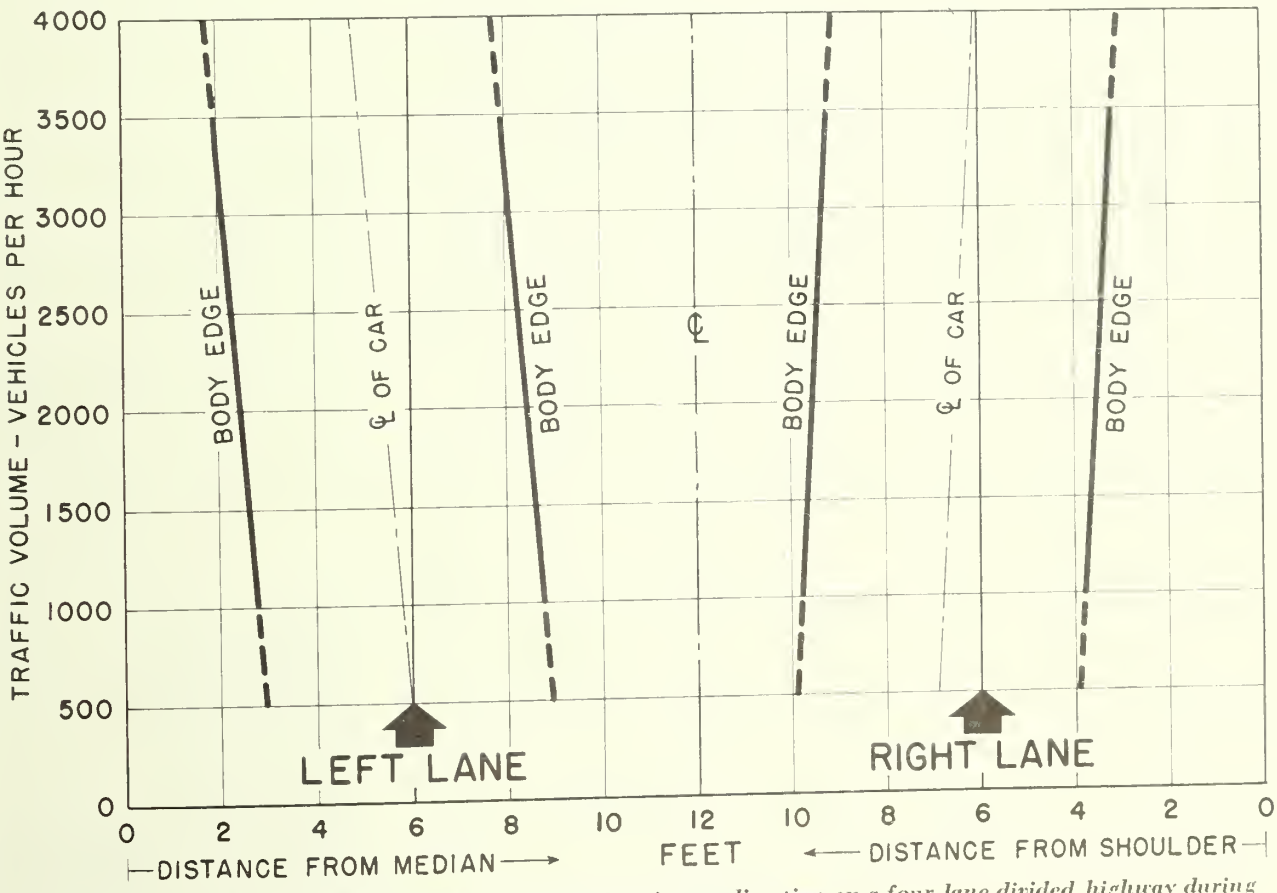


Figure 12.—Average position of passenger cars traveling in one direction on a four-lane divided highway during different traffic volumes.

Aerodynamic Characteristics of a Suspension Bridge

Analysis Based on Tests of a Section Model

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by GEORGE S. VINCENT
Highway Bridge Engineer

This report is presented as typical of the essential procedures currently recommended in the study of aerodynamic characteristics of a proposed suspension bridge design. The investigation included calculations of natural frequencies and wave forms, section model tests to determine the aerodynamic behavior of the section, and analysis of the model test data to determine probable behavior of the bridge itself in a uniform steady wind.

THE SUSPENSION bridge over the Missouri River at the Paseo, Kansas City, Mo., was designed by Howard, Needles, Tammen, and Bergendoff, consulting engineers. Since it was proposed to use Federal-aid highway funds to help finance the construction, a conference, attended by representatives of the State Highway Department and the Bureau of Public Roads, was held to consider design features. Some thought was given to the matter of aerodynamic stability. It was considered improbable that this heavy, self-anchored bridge would be much affected by the wind, particularly in view of its close resemblance to the Cologne Bridge over the Rhine River in Germany (built in 1929; blown up in 1945). However, advantage was taken of the availability of a section model which could be readily modified at small cost to represent this bridge, and the investigation here reported was made. Although somewhat curtailed, it involves most of the essential procedures currently recommended for this type of investigation as developed in the cooperative research conducted by the Washington Toll Bridge Authority, the University of Washington, and the Bureau of Public Roads.

Scope and Conclusions of Study

The investigation consisted of three parts: Calculations to determine the natural frequencies and wave forms of the vertical and torsional oscillations in the first symmetric and first asymmetric modes; section model tests to determine the aerodynamic behavior of the section; and analysis of section model data to determine the probable behavior of the bridge itself in a uniform steady wind.

As a result of these studies, the following conclusions were reached:

1. The lowest natural frequencies of the bridge occur in the first symmetric mode and are:

Vertical mode	21.8 cycles per minute.
Torsional mode	37.5 cycles per minute.

2. Tendencies to restricted torsional and vertical oscillations shown by the section model at low wind velocities are so weak that, considering the probable nonuniformity of the wind, they will likely be completely inhibited by the structural damping of the bridge.

Catastrophic torsional oscillation is indicated in a *uniform and steady* horizontal wind beginning at a critical velocity of about 63 miles per hour and reaching double amplitudes of about 6° at 85 to 95 miles per hour, depending upon the structural damping to be expected. If the steady uniform wind angles upward as much as 6° the oscillation would be expected to occur at velocities about 5 miles per hour lower.

3. Closing the sidewalk gratings (but not the slot under the steel angle curb) should increase the critical velocity in a horizontal wind nearly 15 miles per hour; similarly closing both the sidewalk and median gratings should raise it nearly 20 miles per hour.

4. Considering the probable nonuniformity and unsteadiness of the wind at the site, it is expected that only small motions of the bridge are likely to occur.

Description of Bridge

The structure is a self-anchored suspension bridge having main and side spans of 616 and 308 feet, respectively, two 26-foot roadways, two 3-foot open-grating sidewalks, and a 4-foot open-grating median strip 9 inches high. The stiffening girders, which also resist the horizontal pull of the cables, are continuous from anchorage to anchorage and consist of steel box girders 10 feet deep and 3½ feet wide. They are fixed to the steel tower at one main pier and are free to move longitudinally at the three other supports. The cables are of preformed strands each cable having a net area of 72 square inches and a main span sag of 70 feet. Cables and girders are 65.5 feet center to center. The 7-inch concrete floor slab is carried on 21-inch I-beam stringers resting on 6-foot plate girder floor beams at 25-foot, 8-inch centers. K-type lateral bracing is used in the plane of both flanges of the floor beams.

Computed Natural Modes

Table 1 indicates the frequencies of the first symmetric and asymmetric vertical and torsional modes of oscillation as computed under the different assumptions indicated.[1]

Due to the relatively large ratio of side-span length to main-span length (0.5) these symmetric modes have much lower frequencies than do the asymmetric modes and are the motions to be expected if the wind can cause

[1] *Aerodynamic stability of suspension bridges with special reference to the Tacoma Narrows Bridge.* University of Washington Engineering Experiment Station, Bulletin 116, Part II: *Mathematical Analyses*, by F. C. Smith and G. S. Vincent.

The mathematical theory of vibration in suspension bridges, by Bleich, McCullough, Rosecrans, and Vincent. Chapters 4 and 5. Bureau of Public Roads. (Published by Government Printing Office.)

Table 1.—Frequencies of modes as computed from bridge properties

Assumption	Frequency in c. p. m.			
	Symmetric		Asymmetric	
	Vertical	Torsional	Vertical	Torsional
Neglecting continuity of girder	21.8	----	36.3	----
Neglecting continuity and torsional stiffness	----	29.7	----	49.5
Considering continuity of girder	21.8	----	36.3	----
Considering torsional stiffness of girders but not of lateral system	----	34.4	----	58.8
Considering torsional stiffness of girders acting with top and bottom lateral systems	----	[1] 37.5	----	63.3

[1] This figure would be increased about 10 percent if the effect of the torsional rigidity of the towers were also considered.

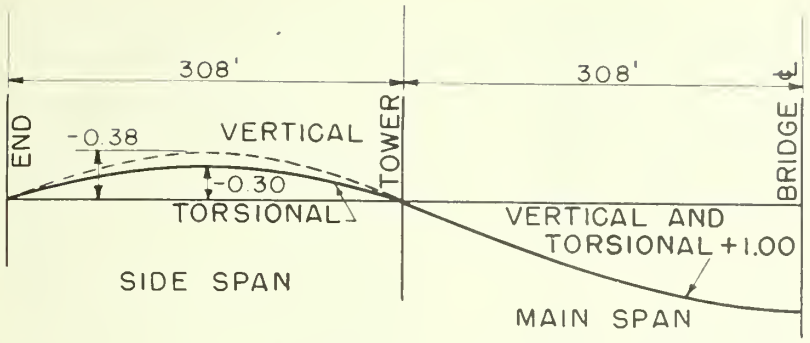


Figure 1.—Wave forms, first symmetric modes.

cillation of the section. The frequencies 21.8 and 37.5 c. p. m. (cycles per minute) in rtical and torsional oscillation reflect the luence of vertical and torsional stiffness and continuity. The other frequencies were mputed to show the effects of the different ffness factors.

The fact that the frequencies of the vertical odes are not affected by girder continuity dicates that the amplitudes of the main-span d side-span waves are such that there is actically no moment at the tower. Mr. J. arol of Howard, Needles, Tammen, and rgendoff has calculated the frequency of the mmetric torsional mode as influenced by the sional resistance of the towers, in addition the full torsional stiffness of the suspended ucture. The tower stiffness increased the quency about 10 percent.

Figure 1 shows the computed wave forms of e first symmetric vertical and torsional odes. The asymmetric wave forms were t computed. They would be essentially e curves over the main span with single-ve curves of opposite signs on the side spans ue to the continuity of the trusses).

The ratio of torsional to vertical frequency the first symmetric modes is 37.5/21.8=1.72. is ratio would be important on a bridge tion subject to pronounced flutter involving coupling of vertical and torsional modes of ilar wave form, such as was found on dels of the proposed design for the Tacoma rrows Bridge before the roadway slots and ttom laterals were provided because, for h a bridge, the velocity required to cause tter increases sharply with an increase in ratio of frequencies. It is of no special nificance on sections showing only weak dencies to coupling, characteristic of girder-fened sections, including the Kansas City dge covered by this report.

Section Model Tests

A comparison of dimensions showed that the ailable 1/50 scale section model of the onx-Whitestone Bridge (as originally built) uld be quite readily modified to represent isfactorily a 1/46.3 scale section model of proposed Kansas City Bridge.[2] Figure 2 shows a cross section of the modified model with dimensions and make-up. Through a misunderstanding, the girder of the model was made to represent the shape of a prototype girder 4 feet wide instead of 3½ feet. This discrepancy was not considered critical and the tests were completed on the model as built. Some other dimensions of the existing model are not in the exact ratio of 1:46.3 to the prototype, but these discrepancies are of no consequence.

The cables are not included in making a section model because full model tests have shown that the wind forces on the cables have no appreciable effect on the oscillation of the structure as a whole. However, the mass of the cables moving in their vertical plane is important. Therefore, it is the practice to increase the weight and mass moment of inertia of the section model by the equivalent of the weight of the cables acting vertically in their planes.

The mass radius of gyration of the bridge cross section as computed by the designer is 24.05 feet, including the cables and hangers acting vertically in their planes. This corresponds to 24.05/46.3=0.52 foot or 6.24 inches on the model. The computed radius of gyration of the model, including attachments, recording pick-up and one-third of the springs, was 6.58 inches, which was considered satisfactory, especially since the ratio of torsional to vertical frequency can be independently controlled on the model.

The dead load of the bridge was given as 12,000 pounds per lincar foot, including the cables. This would require a model weight per foot of model of $12{,}000/46.3^2=5.6$ pounds. The actual weight of the model and fittings per foot of model was $(19.6/61.62)\times 12=3.82$ pounds. Thus the model was deficient in weight in the ratio of $3.82/5.6=0.68$. This means that while the wind forces acting on the model were to true scale as compared with the prototype, being dependent on size and shape, the inertial forces of the model in motion were only 68 percent of what they should have been. The correction which this required when the model results were applied to the bridge is explained in the last section of this report, where the prototype response is analyzed.

The model was suspended from four helical springs. These are usually designed to reproduce to scale the frequency of the prototype vertical mode under investigation (in this case, $21.8\sqrt{46.3}=148$ c. p. m.). For these tests however, the most suitable available set of springs, giving a frequency in still air of 68.3 c. p. m., was used. This was permissible because of the practically constant ratio of the frequency of an oscillation to the wind velocity required to excite it, a relation found in previous tests and confirmed for this model, as will be shown later.

In the absence of any torsional spring action, the frequency of the torsional mode of the section model would be that of the vertical mode multiplied by $b/2r$, where r is the mass radius of gyration and b the width (center to center of springs in this case). This would give a torsional frequency of $(68.3\times 17.8)\div(2\times 6.58)=92.4$ c. p. m. for this model. However, type I (see fig. 3) restraining wires were used to produce torsional frequencies of 99.7 and 120.5 c. p. m. for two series of tests. Restraining wires, used primarily to prevent swaying of the model in the wind stream, are customarily anchored some 10 feet upwind and downwind from the model. It will be noted that with type II restraining wires, the vertical component of the force in the wires is negligible whether the model

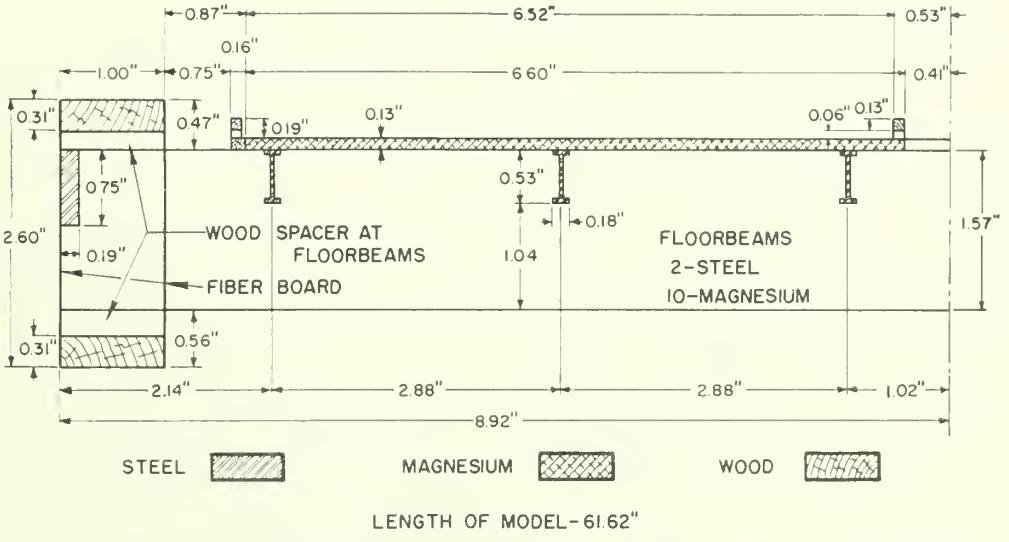
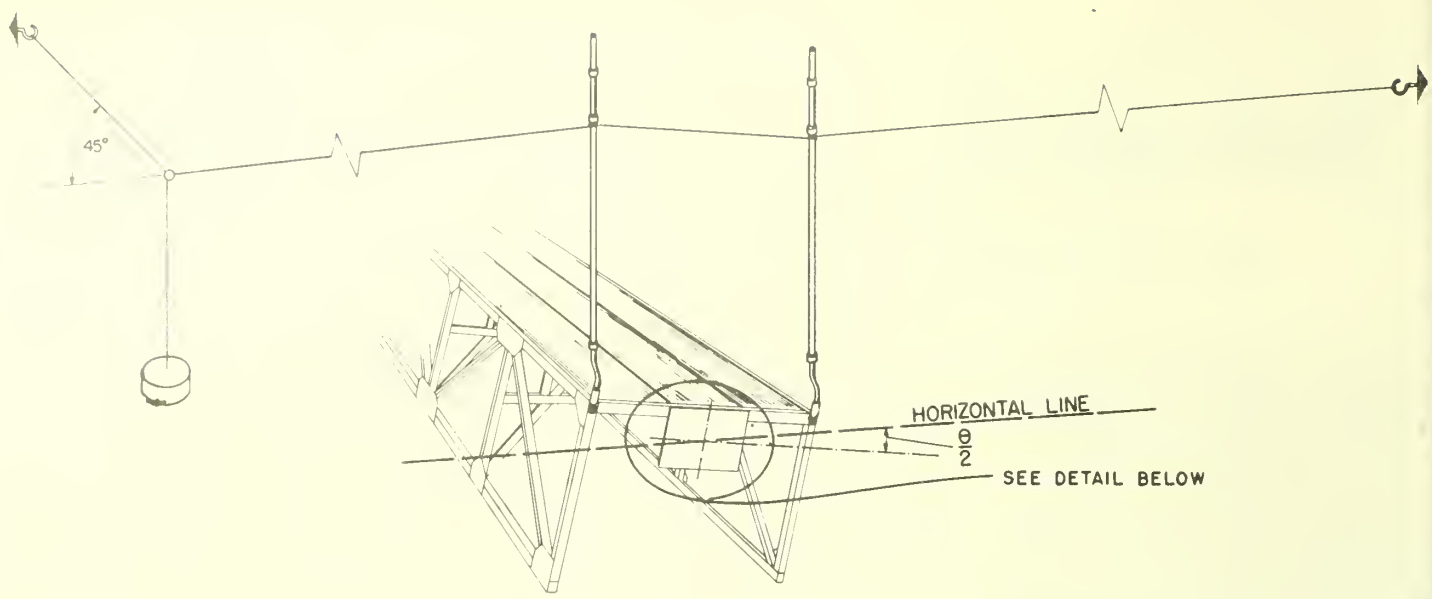
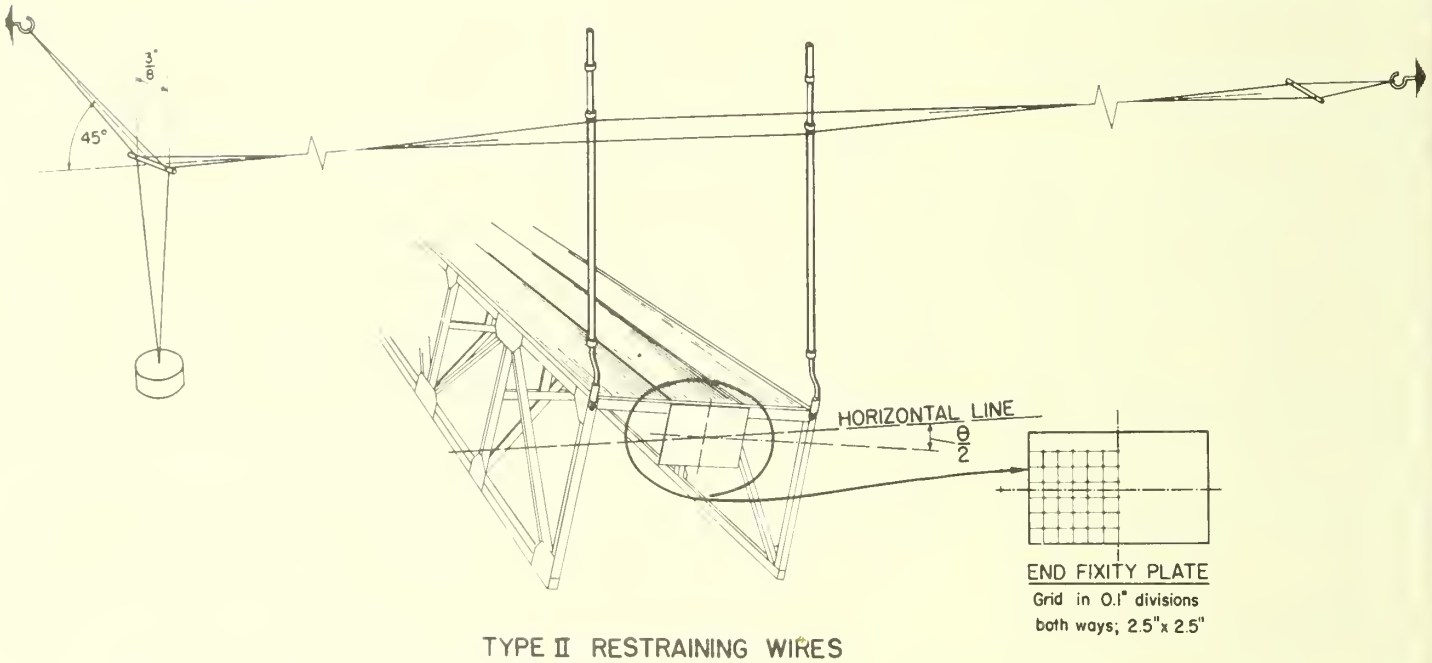


Figure 2.—1/46.3 scale section model of a suspension bridge over the Missouri River, Kansas City, Mo.

Tests on section models of the Bronx-Whitestone Bridge the Paseo Bridge, as well as several others, are reported *lerodynamic stability of suspension bridges with special ref-ce to the Tacoma Narrows Bridge.* University of Wash-on Engineering Experiment Station, Bulletin 116. t V: *Extended studies: logarithmic decrement, field damping, otype predictions, four other bridges,* by G. S. Vincent.



TYPE I RESTRAINING WIRES



TYPE II RESTRAINING WIRES

Figure 3.—End fixity plates, required for tests on some models, were not used on the Paseo Bridge model.

oscillates in vertical or torsional motion. With the type I restraining wires, this is still true in the case of vertical motion, and is true for torsional motion with respect to the portion of the wire from each end to the model support. However, that portion of the wire between the model supports will have a very significant vertical component in torsion. Thus, the type I wires act as springs to restrain torsional, but not vertical, motion and their effect is to increase the frequency of the torsional oscillation, while having little influence on that of the vertical. The strength of this action is increased by increasing the tension of the restraining wire.

Tests were made in a horizontal wind and at an angle of attack β of $+6°$ (wind angled upward 6°) and $-6°$ (wind angled downward 6°).

A test series was run with the sidewalk grating closed except for a small slot under the curb, and a partial series was run with both the sidewalk and median-strip gratings similarly closed.

Test Data

The solid lines in figure 4 show the amplitude of vertical and torsional oscillation plotted against the wind velocity for the model mounting that gave natural (wind off) frequencies of 68.3 and 99.7 c. p. m. in vertical and torsional oscillation, respectively. It will be noted in this figure that the wind-forced frequencies were only slightly greater in vertical motion and slightly less in t- sional motion than the respective natu l frequencies, which indicates that the coupli g tendency—the wind-induced interaction f torsional and vertical motion—was sm . This was true in all of the tests on this mo l and is, in general, characteristic of gird - stiffened sections with their blunt edges.

Previous tests on these springs and th r mountings had indicated a logarithmic d - rement δ_s due to structural damping alo e of about 0.002 on the model, which is, f course, very low compared to the dampi g to be expected on a bridge.

The solid lines in figure 5 show the respo e curves obtained after the tension in the - straining wires had been increased to give a

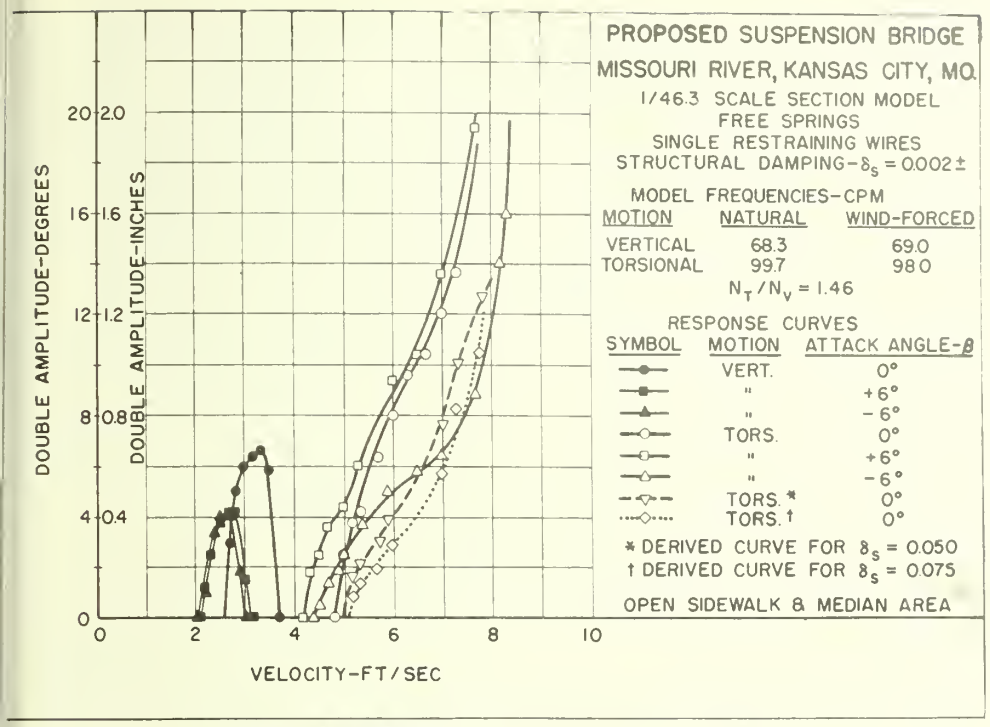


Figure 4.—Model response curves, $N_T/N_V=1.46$.

atio of torsional to vertical frequency of 1.73. The added tension raised the frequency 21 ercent in the torsional mode and only 2 per- ent in the vertical mode.

It will be noted in both figures 4 and 5 that he vertical oscillation due to a horizontal vind was restricted to the narrow velocity ange from about 2¼ to 3½ feet per second nd reached a maximum double amplitude f about 0.7 inch over a very restricted range ear 3¼ feet per second. Figure 4 shows that when the angle of attack β was $+6°$ or $-6°$ the vertical oscillation was even less and was restricted to a narrower velocity range.

The torsional oscillation, on the other hand, began at a fairly distinct "critical velocity" but increased in amplitude as the velocity was increased and showed no tendency to drop off at higher velocities within the scope of the tests. (The test records for $\beta=0°$, fig. 4, show that the double amplitude exceeded 26° for velocities from 7.77 to 15.06 feet per second.) The model was seized to prevent greater motion, which would have caused damage. It will be noted in both figures 4 and 5 that the torsional oscillation began at a lower critical velocity when the angle of attack was changed from 0° to +6°. Also, the critical velocity for a given angle of attack was increased when the frequency of the torsional oscillation increased (compare figs. 4 and 5).

The solid lines in figure 6 show the amplitudes in vertical and torsional oscillation plotted against the ratio, V/Nb, in which V is the velocity in feet per second, N is the frequency in cycles per second, and b is the width of the model in feet. The curves plotted from model tests made at different frequencies fall so close together as to verify for this section the statement that the wind velocity required to cause a given oscillation varies in direct proportion to the frequency of the oscillation. The dimension b of the model is introduced in the denominator in order to make the ratio nondimensional and thus applicable to the prototype or any scale model of it. Thus, knowing from model tests the critical value of the V/Nb ratio, designated $(V/Nb)_c$, at which a wind-excited oscillation (vertical or torsional) will begin, the wind velocity V_p at which the prototype will begin to oscillate can be determined in feet per second from the equation:

$$V_p/N_p b_p=(V/Nb)_c \quad \text{(1)}$$

by substituting the known frequency N_p and width b_p of the prototype.

Prediction of Prototype Behavior

Whether or not an oscillation observed on the model will appear on the prototype in an ideal wind stream (uniform and steady) at the prototype frequency, amplitude, and wind velocity will depend primarily upon the relative strengths of the exciting wind forces and the damping forces. This cannot be determined directly from the model response curves unless the structural damping of the model set-up happens to be fairly close to that of the prototype, which would only be true by accident, since it is impracticable to attempt to reproduce to proper scale on a model all of the damping influences of a bridge. However, a technique has been developed whereby the structural damping of an existing suspension bridge can be determined with reasonable accuracy by means of dynamic tests on the bridge and wind-tunnel tests on a section model of it, together with a correlating theoretical analysis. The strength of the exciting force, considered mathematically as negative damping, can be determined from the oscillograph records of the motion of the model.

A convenient measure of the strength of the exciting and damping forces is their ability to increase or decrease the amplitude of a vibration. This may be expressed by the logarithmic decrement δ, which is the natural logarithm of the ratio of the peak amplitudes of two successive cycles; that is,

$$\delta=\log_e\frac{\eta_0}{\eta_1} \quad \text{(2)}$$

in which η_0 and η_1 are, respectively, the amplitude peaks of any cycle and of the cycle immediately following it. For a vibration acted upon only by viscous damping (a damping force proportional to the velocity of vibration), the amplitude decays as a logarithmic or exponential curve and the value of δ is constant for all amplitudes.

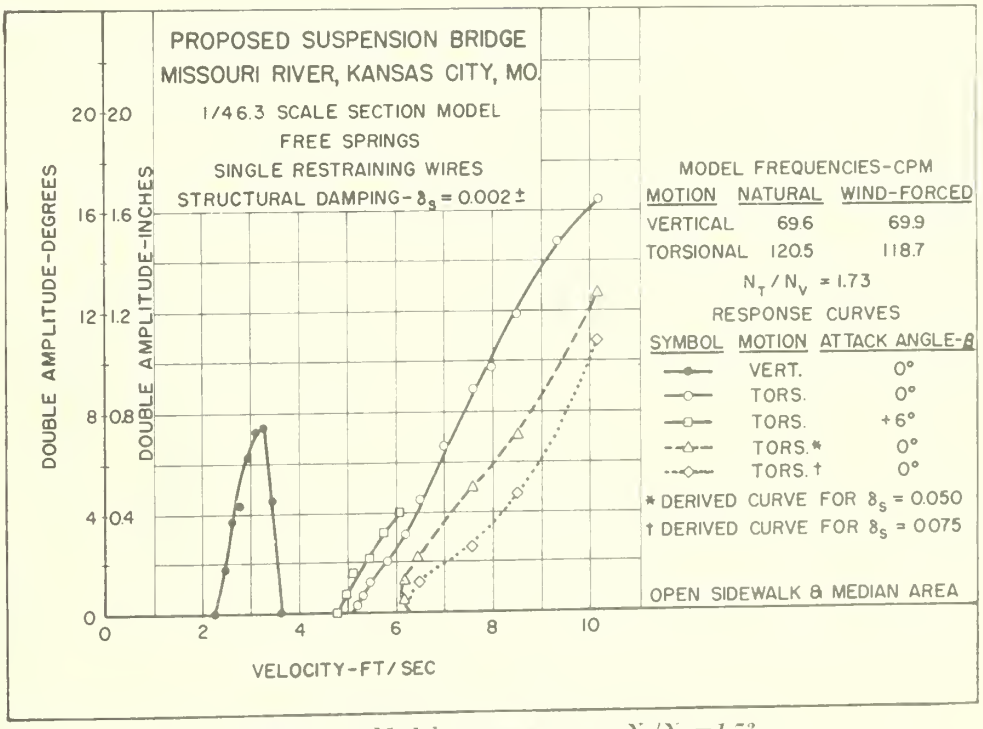


Figure 5.—Model response curves, $N_T/N_V=1.73$.

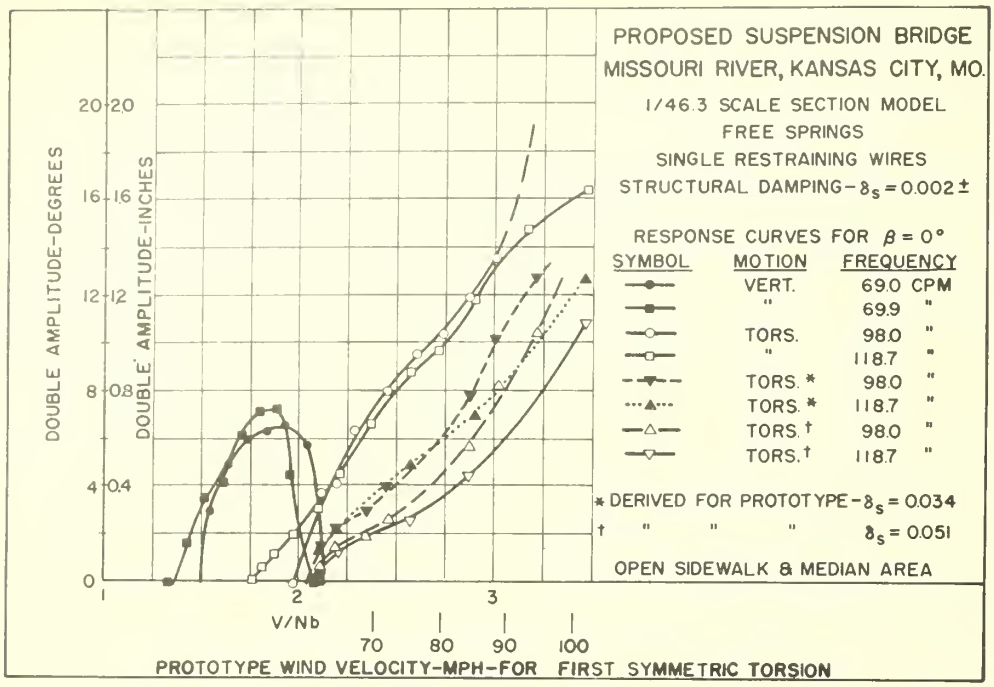


Figure 6.—Model response plotted against V/Nb ratio; also predicted prototype response.

Many forms of damping are not purely viscous, so that the value of δ varies when computed at different points along the curve of amplitude decay. This is true of the aerodynamic damping in still air and especially true of the complex damping due to a wind of constant velocity, which not only varies with the amplitude but may be negative over certain amplitude ranges and positive over others.

The damping affecting the vibration of a model or bridge always includes both the structural damping of the mechanism and the damping due to motion in still air or in a wind. This may be expressed as $\delta=\delta_s+\delta_a$ in which the subscripts $_s$ and $_a$ designate the structural and aerodynamic damping, respectively. The plotted points on response curves such as shown in figures 4 and 5 represent the amplitude which will build up and remain steady when the velocity is maintained constant at the value indicated. At any steady-state amplitude the value of δ must be zero (since $\eta_1=\eta_0$), which can only be true when the negative value of δ_a is just equal numerically to the positive value of δ_s.

The values of δ were determined from the measured amplitudes of the oscillograph record taken while the motion built up from rest to the steady state for several of the wind-velocity test runs used in plotting the response curves for $\beta=0°$ in figures 4 and 5. The values of δ_a were determined by subtraction of 0.002, the approximate value of δ_s, from δ determined from the records, and δ_a was plotted against amplitude in figures 7 and 8, each curve being identified by the wind velocity for which it was determined and by the value of the V/Nb ratio corresponding to that velocity.

The intersections of the decrement curves with the amplitude axis, representing $\delta_a=0$, mark the steady-state amplitude which would be reached at the corresponding velocities if there were no structural damping. If the structural damping δ_s is plotted downward from the axis of zero damping, its intersections with the aerodynamic decrement curves will mark the amplitudes at which $\delta_s=-\delta_a$ or $\delta_s+\delta_a=0$, thus indicating the steady-state amplitudes.

The short- and long-dash horizontal lines in figures 7 and 8 represent $\delta_s=0.05$ and $\delta_s=0.075$, respectively. The amplitudes indicated by their intersections with the aerodynamic decrement curves were plotted in figures 4 and 5 to produce, respectively, the dotted and dashed response curves.

Effect of Underweight Model

Earlier in this report it was mentioned that a correction would be required in applying the model test results to the prototype because the model weight was only 68 percent of that of a truly scaled model. It was noted that the disturbed the relation between the wind forces, dependent upon size and shape, and the inertial forces, dependent upon mass.

Most reference books on vibration show the logarithmic decrement in terms of other properties of the vibrating system, thus

$$\delta=\frac{cg}{2wN} \qquad (3)$$

in which c is the damping force per unit velocity, g is the acceleration of gravity, w is the weight, and N is the frequency. In the tests, the aerodynamic damping force corresponding to c in the expression for δ was correctly scaled but the weight w was only 68 percent of the correct value. It follows that δ_a as determined from the tests was too large and should be multiplied by 0.68 to obtain the correct value. This correction could be made graphically in figures 7 and 8 by simply multiplying the ordinate scale by 0.68. If this were done the ordinates of the short- and long-dash curves for $-\delta_s$ would represent values of 0.034 and 0.051 instead of 0.05 and 0.075. The steady-state amplitudes indicated by the intersections of these curves with the aerodynamic decrement curves were therefore plotted against the V/Nb ratio in figure 6 and identified as derived curves for the prototype for $\delta_s=0.034$ and $\delta_s=0.051$. The prototype wind velocity scale corresponding to the

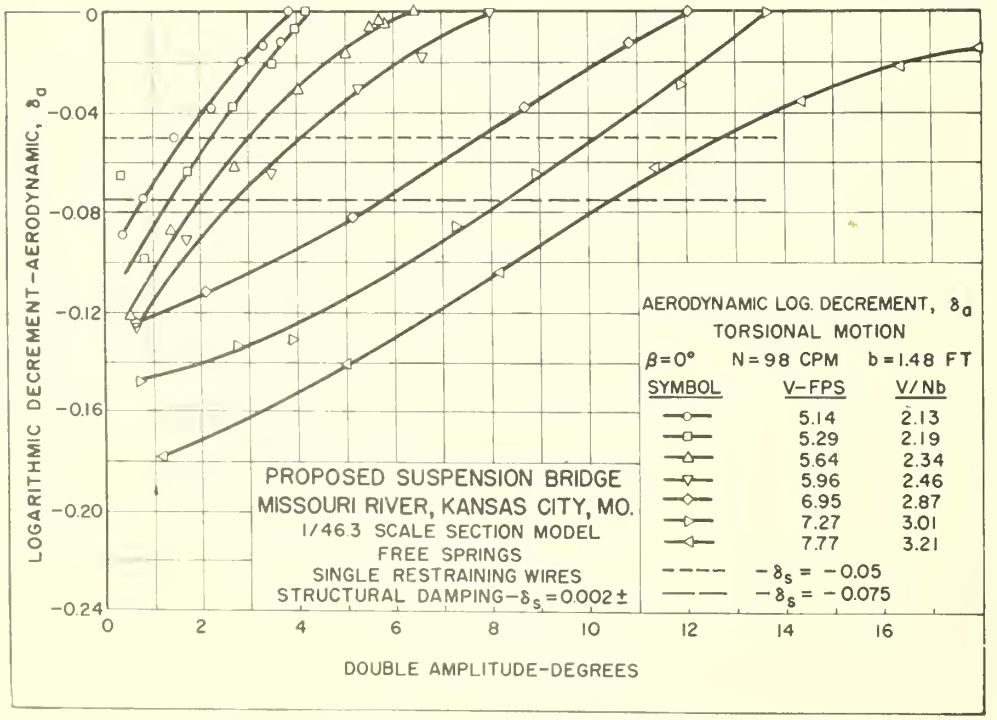


Figure 7.—Decrement curves for torsional oscillation at 98 cycles per minute.

[3] For a fuller discussion of damping in relation to suspension bridges see Appendix II.—*Energy and damping*, in *aerodynamic stability of suspension bridges: progress report of the advisory board on the investigation of suspension bridges.* Proceedings of the American Society of Civil Engineers, vol. 78, Separate Nos. 144A, 144B, Aug. 1952.

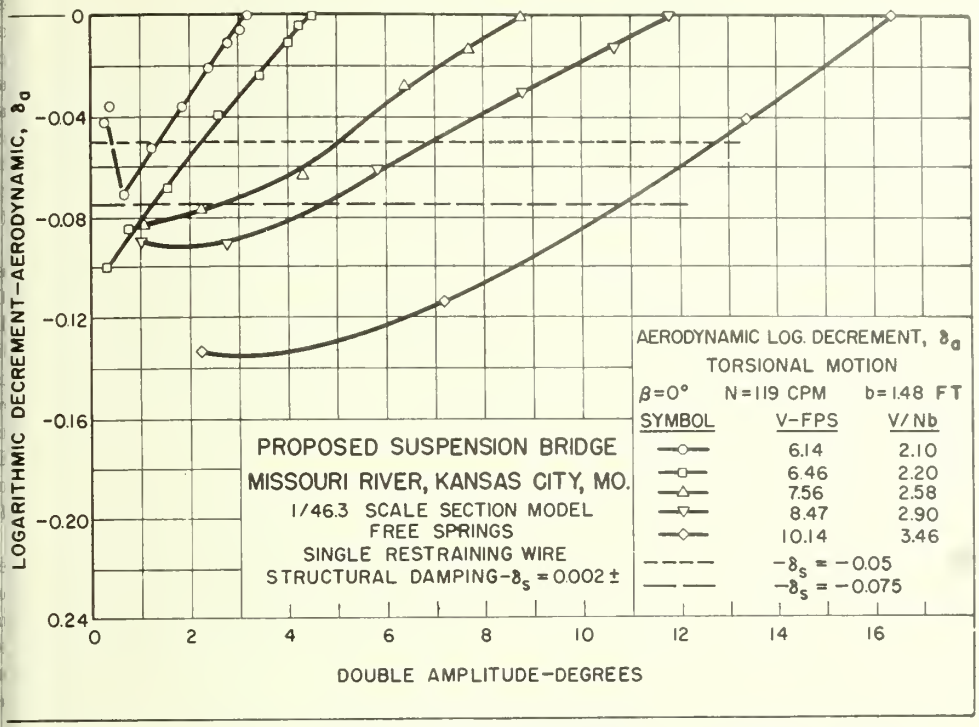


Figure 8.—Decrement curves for torsional oscillation at 119 cycles per minute.

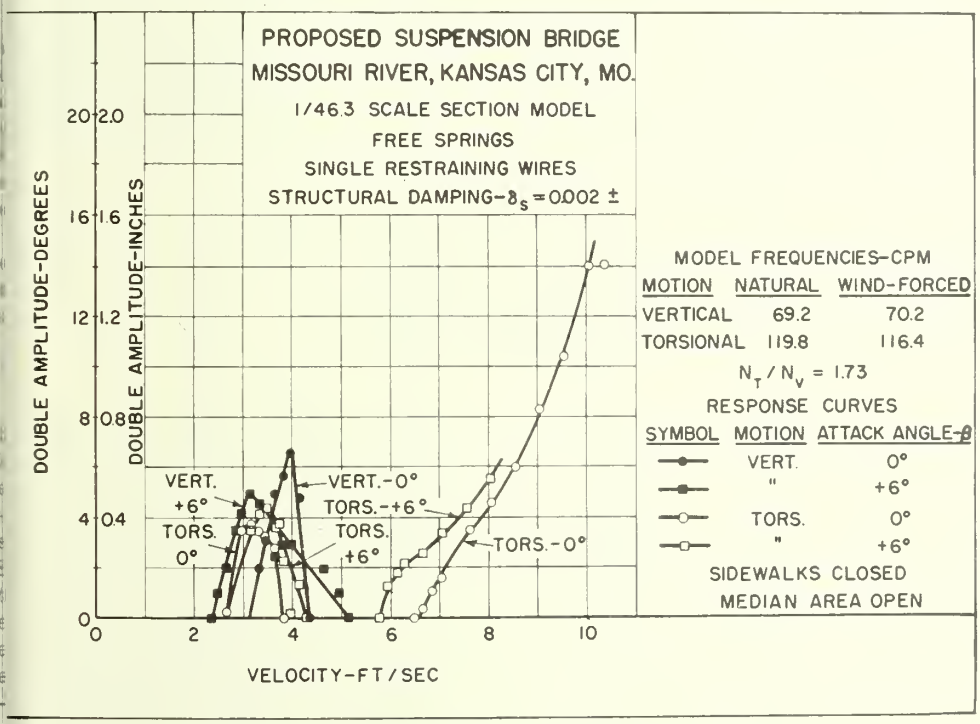


Figure 9.—Response curves with sidewalk grating closed.

V/Nb scale for the first symmetric torsional mode is shown in figure 6. It was computed from equation (1), using the prototype values N_p=37.5 c. p. m. and b_p=69 feet, thus:

$$V_p=(V/Nb)N_p b_p=\frac{37.5}{60}\times 69(V/Nb)$$

$$=43.12(V/Nb) \text{ feet per second}$$

$$=29.4(V/Nb) \text{ miles per hour.}$$

Probable Behavior of Bridge

Torsional oscillation.—There is evidence that δ_s for the Golden Gate Bridge and the original Tacoma Narrows Bridge is well below 0.05. By comparison with these, the bridge at Kansas City, with a much greater proportion of the total energy stored in bending and twisting the heavy girders and with top and bottom lateral systems forcing more participation of the concrete deck, should have a structural decrement somewhat more than 0.05. Using the predicted response curve for δ_s=0.051 in figure 6, the first symmetric torsional mode would be expected to appear at a velocity of about 62 miles per hour and reach about 6° of double amplitude at 85 to 90 miles per hour in a *steady uniform wind.* Unless the wind is unusually steady and uniform along the bridge, however, it is likely that even maximum storms will cause only small, more or less spasmodic oscillation, perhaps enough over a few cycles to be identified as to frequency and general wave form.

Vertical motion.—Figure 6 shows the maximum amplitude of the vertical oscillation of the section model occurring at about V/Nb= 1.85. For the first symmetric vertical mode of the prototype N_p=21.8 c. p. m. and the peak amplitude would be expected at:

$$V_p=1.85\times\frac{21.8}{60}\times 69\times\frac{3,600}{5,280}=31.6 \text{ miles per hour}$$

However, variations of only a few miles per hour above or below this figure would greatly reduce the effectiveness of the wind ($-\delta_a$ was generally less than 0.04 and reached 0.06 to 0.07 within a very narrow velocity range).

Although δ_s for the vertical motion may be less than 0.05, it is unlikely that vertical oscillation will appear except in random form or for a few cycles.

Effect of closing slots.—Figure 9 shows model response curves with the sidewalks covered but with a small gap under the curbs. By comparison with the solid-line curves of figure 5, it is seen that closing the sidewalk grating raised the critical velocity of the torsional mode about 1.0 foot per second for $\beta=+6°$ and about 1.5 for $\beta=0°$. The increases in $(V/Nb)_c$ are about 0.3 and 0.5, corresponding to 10 to 15 miles per hour for the first symmetric torsional mode of the prototype.

Figure 10 shows the response at $\beta=0°$ when the sidewalks and median strip were covered except for the small curb slot. Comparing this with figure 5 shows an increase of more than 2 feet per second in critical velocity or over 20 miles per hour on the prototype.

These tests support others indicating that roadway slots which are beneficial in combatting flutter of thin sections are often detrimental in dealing with torsional oscillation of the blunt sections of girder-stiffened suspension bridges. The excitation of the latter arises largely from the wind flow adjacent to the girders, while the roadway slots seem to reduce the damping effect of the deck.

Acknowledgment

The section model tests were made in the suspension bridge wind tunnel at the University of Washington under the cooperative agreement of the University and the Bureau of Public Roads covering the studies on the Tacoma Narrows Bridge and a number of other suspension bridges. Professor F. B. Farquharson, Director of the Engineering Experiment Station, approved the added tests on behalf of the University. Robert E. McHugh, Jr., Junior Research Engineer for the University of Washington, made the tests. The writer, representing the Bureau of Public Roads, computed the vibration characteristics of the bridge, made the required modifications in the model, and analyzed the data. The designers of the bridge, Howard, Needles, Tammen, and Bergendoff, also checked the vibration calculations for this report.

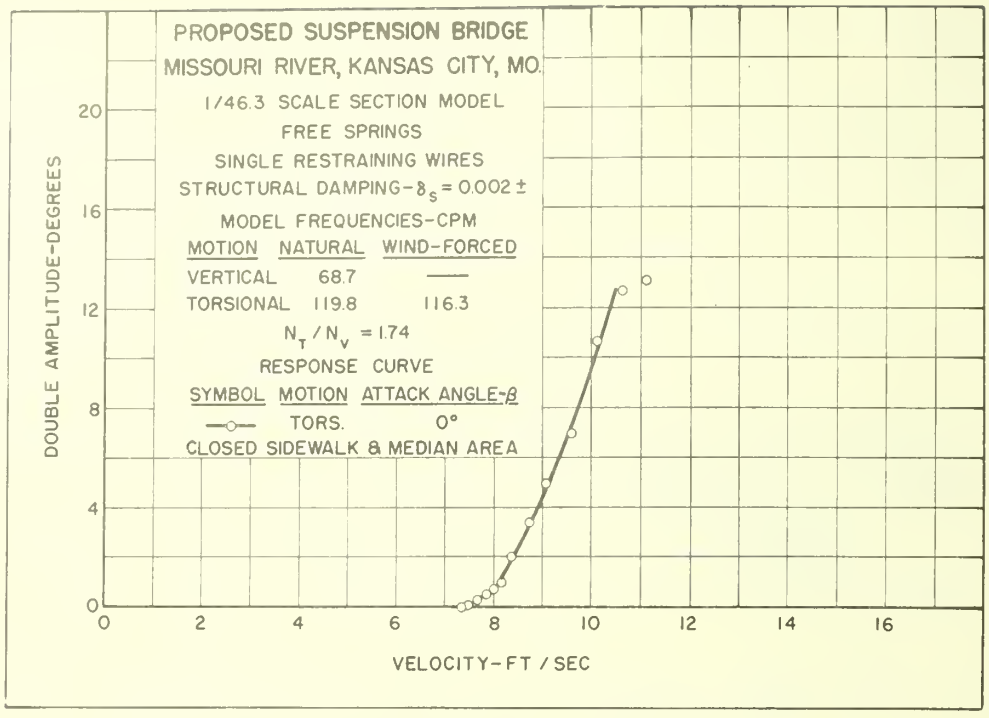


Figure 10.—Response curve with sidewalk and median area gratings closed.

Driver Behavior

(Continued from page 169)

vehicles traveling in the right lane were affected approximately the same as passenger cars by objects at the pavement edge. At the higher volumes, the effect on commercial vehicles was slightly less than on passenger cars. The lateral positions of commercial vehicles were not influenced by a barricade placed either 3 or 6 feet from the pavement or by a truck or car parked 6 feet therefrom.

Although the lateral positions of vehicles in the right lane were affected by objects on the shoulder, the objects did not cause change in the distribution of traffic betwe the two lanes. The objects on the should did, however, cause an increase in the pe centage of commercial vehicles that straddl the lane marking

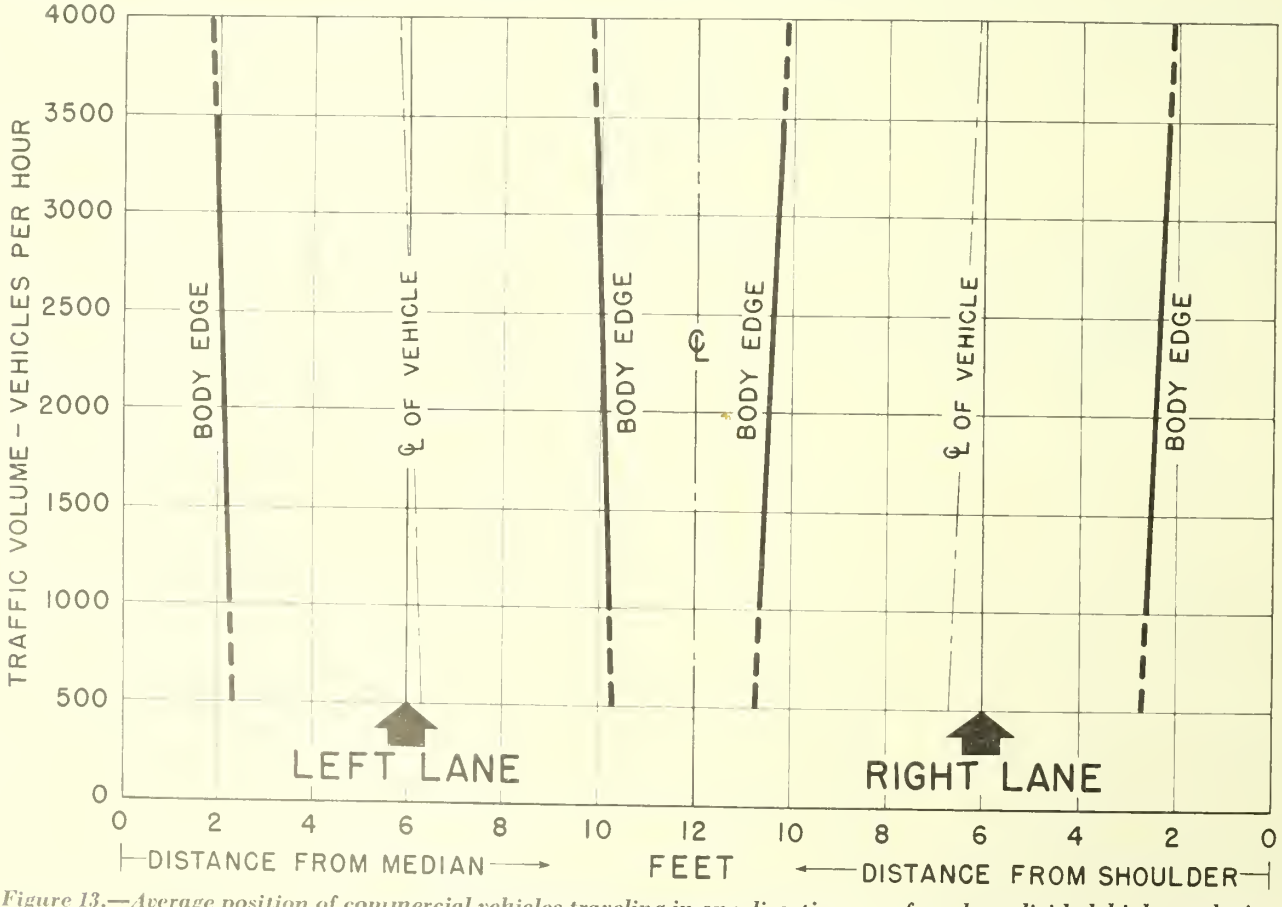


Figure 13.—Average position of commercial vehicles traveling in one direction on a four lane divided highway during different traffic volumes.

Needs of the Highway Systems, 1955-84

On March 25, 1955, the Secretary of Commerce transmitted to the Congress a report on the cost of construction needed to modernize the Nation's highways, prepared by the Bureau of Public Roads in cooperation with the State highway departments, pursuant to the requirement in section 13 of the Federal-Aid Highway Act of 1954.

This report, titled *Needs of the Highway Systems, 1955–84,* has been published as House Document No. 120, 84th Congress, 1st Session, and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 5 cents a copy.

The cost of construction needed to modernize the Nation's roads and streets within the 30-year period 1955–64 would amount to $101 billion, the report shows. Construction needs for the following 20 years, 1965–84, to sustain adequacy and provide for expanding traffic growth, were estimated to cost $114 billion.

Maintenance needs and administrative costs must also be considered in the total cost of a highway program. Maintenance needs for the initial 10-year period were estimated to total $19.4 billion, and for the 1955–84 period, $48.8 billion. The estimates of administrative costs were $5.9 billion for the first 10-year period and $7.8 billion for 1965–84.

The total estimated needs for the 30 years from 1955 to 1984 thus amount to $297.1 billion, of which 72 percent is for construction. The total needs over the entire period average out to $9.9 billion a year. By way of comparison, the total expenditure (exclusive of debt service) for all roads and streets in 1954 was $6.1 billion, of which 64 percent was for construction.

The several estimates were made for five major systems: Interstate, other Federal-aid primary, Federal-aid secondary, other State highways, and all other roads and streets. Each of these was divided into rural and urban except the Federal-aid secondary system, which is almost wholly rural. The estimates for this system were divided according to State or local control. The estimated construction needs for the initial 10-year period were reported by States, and these data are reproduced as table 1 on page 178.

In a statement on highway financing included in the report, it is noted that the established flow of funds for highways at current tax rates, taking into account increasing motor-vehicle usage, should produce $47 billion for highway construction in the 10-year period 1955–64. A deficit of $54 billion must be overcome if the $101 billion needs are to be met.

Decision as to a financing program should take into account the division of cost burden among various levels of government, the amount that can be spent for highways in view of other public needs, and the suitability of various methods of obtaining funds for highways.

The President's Advisory Committee on a National Highway Program, headed by Gen. Lucius D. Clay, presented its recommendations in a January 1955 report, *A Ten-Year Highway Program* (available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 20 cents per copy). The Committee proposes a total construction expenditure by the Federal Government of $31.225 billion over the next 10 years. Of the total, $25 billion is for the interstate system including essential urban arterial connections, $3.15 billion for the remainder of the Federal-aid primary system, $2.10 billion for the Federal-aid secondary system, $0.75 billion for the Federal-aid urban system, and $0.225 billion for forest highways. Financial participation by State and local governments would amount to $2 billion on the interstate system including essential urban arterial connections. For the other Federal-aid systems, statutory matching requirements would remain unchanged and would amount to slightly less than the Federal contributions of $6.225 billion.

Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program

The Secretary of Commerce transmitted to the Congress on April 14, 1955, a report on the progress and feasibility of toll roads and their possible effects on the Federal-aid highway program, in accordance with section 13 of the Federal-Aid Highway Act of 1954. The study was made by the Bureau of Public Roads in cooperation with the State highway departments.

The report, entitled *Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program,* has been published as House Document No. 139, Eighty-fourth Congress, first session, and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents a copy.

The study found that highways feasible of construction as toll roads lie almost entirely within the 40,000-mile network of the National System of Interstate Highways. This interstate system, linking industrial and commercial centers from coast to coast, embraces all but about 200 miles of the 6,900-mile total which the report regards as now feasible for additional toll-road construction.

In making this estimate of toll-road feasibility the investigation excluded routes on which present improvements or those already programed for early construction were expected to meet traffic demands of the future. Thus the toll-road potential is in addition to mileage, either toll or free, that now meets traffic requirements and those sections which definitely programed improvements will make satisfactory.

Feasibility of potential toll roads was determined by dividing the estimated yearly net operating income for a particular section of road by the annual cost of debt service resulting from the issuance of revenue bonds. Recent experience shows that bonds offered to finance toll roads when the annual net revenue is one and one-half times the yearly debt service would be marketable. Therefore, any section of road having a revenue-debt service ratio of 1.5 was regarded as feasible for toll-road construction and financing.

The report estimates that the construction cost of the 6,700-mile potential which lies within the interstate system would be $4,260,000,000. Some of this mileage is located in States that have not authorized toll financing and which may not do so in the future. Other sections lie in States where authority to construct toll roads is limited to particular routes and may not include some of the sections which are considered feasible in this report. To the extent that these conditions prevail, the 6,700-mile figure might be reduced.

The greatest uncertainty, however, centers on the policies of the States and the Federal Government with respect to the public funds from taxes or bond issues that will be applied to the improvement of the interstate system. The report points out that assurance of public funds to provide reasonably early completion of this vital network would soon spell the end of revenue-bond financing of roads in the system. If the present inadequate allocation of funds to this system is continued, the mileage of potentially feasible toll roads will increase.

These findings not only show the importance of the interstate system, they also raise questions concerning Federal participation in toll roads. As directed by the Federal-Aid Highway Act of 1954, the report includes recommendations covering this point:

1. *No Federal participation in toll roads.*— The present law forbidding the collection of tolls on highways constructed with Federal-aid funds should be continued.

2. *Inclusion of toll roads in Federal-aid systems.*—The present law should be changed to permit the inclusion of toll roads as part of the National System of Interstate Highways when they meet the standards for that system, *and when there are reasonably satisfactory alternate free roads* on the Federal-aid primary or secondary systems which permit traffic to bypass the toll roads.

This recommendation is made to meet present-day conditions. A number of toll roads which are in operation, under construction, or authorized, lie along the preferred location of interstate routes; duplication of these roads would generally be an economic waste. Accordingly, if there is to be a continuous integrated interstate system, it is reasonable that these toll roads be included in it. The inclusion of a toll road in the interstate system would not be contrary to recommendation 1. It would merely make it unnecessary to construct a free road to interstate standards closely paralleling the toll road.

No toll roads should be permitted on any Federal-aid system except as provided in the first paragraph of this recommendation. Continuous travel over free roads will then be possible except over those portions of the interstate system on which tolls are collected. On those portions, drivers will have the alternative of travel over a toll road built to interstate standards, or over a reasonab satisfactory free road of another Federal-a route.

Some of the adverse effects of toll roads the programing of public highway improv ments and many problems of integration toll roads with public highway networks wou be greatly alleviated if the responsibility f toll roads were vested in State highwa departments. The consolidation of respon bility for toll and free highways would perm the most effective use of available engineeri and technical personnel, avoid duplicati administrative organizations, and promo orderly development of all highway improv ments.

Table 1.—Highway construction needs, 1955–64, by system and State

[All amounts in millions of dollars] [1]

State	Federal-aid Systems												Non Federal-aid systems									All roads and streets		
	Interstate			Other Federal-aid primary			Federal-aid secondary			Total			Other State highways			Other roads and streets			Total					
	Rural	Urban	Total	Rural	Urban	Total	State	Local	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total	Rural	Urban	Total
Alabama	316	50	366	139	153	292	23	111	134	589	203	792	2	11	13	780	469	1,249	782	480	1,262	1,371	683	2,054
Arizona	193	17	210	120	12	132	94	49	143	456	29	485	------	------	------	135	36	171	135	36	171	591	65	656
Arkansas	171	32	203	223	26	249	271	78	349	743	58	801	10	5	15	107	161	268	117	166	283	860	224	1,084
California	592	1,729	2,321	859	677	1,536	360	288	648	2,099	2,406	4,505	351	423	774	1,385	1,545	2,930	1,736	1,968	3,704	3,835	4,374	8,209
Colorado	130	27	157	321	60	381	197	------	197	648	87	735	6	------	6	473	236	709	479	236	715	1,127	323	1,450
Connecticut	185	370	555	237	381	618	166	41	207	629	751	1,380	229	144	373	157	144	301	386	288	674	1,015	1,039	2,054
Delaware	45	21	66	81	26	107	83	------	83	209	47	256	28	2	30	9	13	22	37	15	52	246	62	308
Florida	360	136	496	107	724	831	242	74	316	783	860	1,643	62	158	220	124	43	167	186	201	387	969	1,061	2,030
Georgia	569	131	700	467	174	641	189	166	355	1,391	305	1,696	56	31	87	268	206	474	324	237	561	1,715	542	2,257
Idaho	98	9	107	169	9	178	64	35	99	366	18	384	8	1	9	153	21	174	161	22	183	527	40	567
Illinois	547	518	1,065	726	256	982	31	108	139	1,412	774	2,186	223	182	405	439	1,447	1,886	662	1,629	2,291	2,074	2,403	4,477
Indiana	475	392	867	816	303	1,119	385	362	747	2,038	695	2,733	18	33	51	741	681	1,422	759	714	1,473	2,797	1,409	4,206
Iowa	220	55	275	978	194	1,172	------	514	514	1,712	249	1,961	3	1	4	390	132	522	393	133	526	2,105	382	2,487
Kansas	191	16	207	570	96	666	107	207	314	1,075	112	1,187	------	------	------	277	198	475	277	198	475	1,352	310	1,662
Kentucky	363	129	492	355	132	487	416	79	495	1,213	261	1,474	215	71	286	108	69	177	323	140	463	1,536	401	1,937
Louisiana	246	247	493	159	73	232	256	3	259	664	320	984	215	56	271	66	241	307	281	297	578	945	617	1,562
Maine	140	7	147	219	126	345	140	------	140	499	133	632	105	18	123	84	31	115	189	49	238	688	182	870
Maryland	155	279	434	212	284	496	201	57	258	625	563	1,188	14	14	28	219	89	308	233	103	336	858	666	1,524
Massachusetts	144	694	838	340	285	625	94	318	412	896	979	1,875	9	3	12	361	201	562	370	204	574	1,266	1,183	2,449
Michigan	807	488	1,295	1,385	440	1,825	532	736	1,268	3,460	928	4,388	57	105	162	655	1,496	2,151	712	1,601	2,313	4,172	2,529	6,701
Minnesota	262	221	483	305	189	494	113	116	229	796	410	1,206	2	5	7	198	239	437	200	244	444	996	654	1,650
Mississippi	222	24	246	270	33	303	80	47	127	619	57	676	17	4	21	128	98	226	145	102	247	764	159	923
Missouri	436	163	599	699	178	877	308	------	308	1,443	341	1,784	22	8	30	290	427	717	312	435	747	1,755	776	2,531
Montana	141	12	153	317	4	321	------	79	79	537	16	553	------	1	1	170	25	195	170	26	196	707	42	749
Nebraska	90	16	106	159	31	190	112	57	169	418	47	465	2	2	4	191	161	352	193	163	356	611	210	821
Nevada	63	10	73	83	10	93	20	21	41	187	20	207	2	------	2	3	8	11	5	8	13	192	28	220
New Hampshire	42	24	66	159	45	204	82	2	84	285	69	354	67	8	75	46	83	129	113	91	204	398	160	558
New Jersey	223	1,134	1,357	625	1,081	1,706	17	232	249	1,097	2,215	3,312	22	70	92	344	875	1,219	366	945	1,311	1,463	3,160	4,623
New Mexico	176	59	235	201	46	247	117	------	117	494	105	599	74	23	97	182	128	310	256	151	407	750	256	1,006
New York	638	698	1,336	2,063	1,293	3,356	483	406	889	3,590	1,991	5,581	7	18	25	268	553	821	275	571	846	3,865	2,562	6,427
North Carolina	226	21	247	314	42	356	255	------	255	795	63	858	6	15	21	182	155	337	188	170	358	983	233	1,216
North Dakota	103	4	107	74	8	82	120	68	188	365	12	377	------	1	1	49	57	106	49	58	107	414	70	484
Ohio	537	824	1,361	622	578	1,200	384	91	475	1,634	1,402	3,036	[4] 238	82	320	794	886	1,680	1,032	968	2,000	2,666	2,370	5,036
Oklahoma	296	81	377	505	104	609	117	118	235	1,036	185	1,221	24	13	37	144	137	281	168	150	318	1,204	335	1,539
Oregon	170	149	319	187	146	333	73	33	106	463	295	758	4	18	22	182	109	291	186	127	313	649	422	1,071
Pennsylvania	360	400	760	1,074	482	1,556	1,422	18	1,440	2,874	882	3,756	1,165	85	1,250	433	275	708	1,598	360	1,958	4,472	1,242	5,714
Rhode Island	7	116	123	117	128	245	28	10	38	162	244	406	13	11	24	25	31	56	38	42	80	200	286	486
South Carolina	140	43	183	93	30	123	72	5	77	310	73	383	79	19	98	79	30	109	158	49	207	468	122	590
South Dakota	86	9	95	141	23	164	100	57	157	384	32	416	7	------	7	61	41	102	68	41	109	452	73	525
Tennessee	261	119	380	472	115	587	184	61	245	978	234	1,212	------	7	7	321	260	581	321	267	588	1,299	501	1,800
Texas	515	357	872	520	135	655	464	------	464	1,499	492	1,991	260	29	289	651	743	1,394	911	772	1,683	2,410	1,264	3,674
Utah	189	49	238	100	29	129	48	15	63	352	78	430	39	2	41	104	57	161	143	59	202	495	137	632
Vermont	169	9	178	167	12	179	46	28	74	410	21	431	2	------	2	35	5	40	37	5	42	447	26	473
Virginia	386	183	569	308	159	467	207	21	228	922	342	1,264	1	19	20	283	190	473	284	209	493	1,206	551	1,757
Washington	193	274	467	427	57	484	257	50	307	927	331	1,258	64	33	97	283	331	614	347	364	711	1,274	695	1,969
West Virginia	168	90	258	516	152	668	485	------	485	1,169	242	1,411	------	1	1	304	62	366	304	63	367	1,473	305	1,778
Wisconsin	208	115	323	523	202	725	411	136	547	1,278	317	1,595	------	------	------	366	424	790	366	424	790	1,644	741	2,385
Wyoming	282	14	296	216	10	226	95	10	105	603	24	627	1	------	1	179	24	203	180	24	204	783	48	831
District of Columbia	------	152	152	------	153	153	9	------	9	9	305	314	------	10	10	------	------	------	------	10	10	9	315	324
Subtotal, Continental United States	12,536	10,717	23,253	19,740	9,906	29,646	9,960	4,907	14,867	47,143	20,623	67,766	3,729	1,742	5,471	13,226	13,873	27,099	16,955	15,615	32,570	64,098	36,238	100,336
Hawaii	------	------	------	59	41	100	52	------	52	111	41	152	------	------	------	13	21	34	13	21	34	124	62	186
Puerto Rico	------	------	------	90	46	136	64	------	64	154	46	200	6	23	29	39	14	53	45	37	82	199	83	282
Grand total	12,536	10,717	23,253	19,889	9,993	29,882	10,076	4,907	14,983	47,408	20,710	68,118	3,735	1,765	5,500	13,278	13,908	27,186	17,013	15,673	32,686	64,421	36,383	100,804

[1] Individual entries may vary slightly from original reported data due to rounding in millions of dollars.

U. S. GOVERNMENT PRINTING OFFICE: 1955

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Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.
Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Model Traffic Ordinance (revised 1953). 20 cents.

PUBLICATIONS (Continued)

Motor-Vehicle Traffic Conditions in the United States, House Document No. 462 (1938). Out of print.
Part 1.—Nonuniformity of State Motor-Vehicle Traffic Laws
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Part 3.—Inadequacy of State Motor-Vehicle Accident Reporting.
Part 4.—Official Inspection of Vehicles.
Part 5.—Case Histories of Fatal Highway Accidents.
Part 6.—The Accident-Prone Driver.
Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
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Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
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Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.
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Taxation of Motor Vehicles in 1932. 35 cents.
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Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1955

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,410	$10,778	$5,730	322.9	$4,801	$2,510	48.1	$45,442	$22,874	696.5	$61,021	$31,114	1,067.
Arizona	3,599	6,696	4,281	134.0	1,090	831	39.4	9,021	6,430	150.0	16,807	11,542	323.
Arkansas	7,590	11,948	6,499	406.8	5,007	2,652	66.9	18,027	9,003	511.2	34,982	18,154	984.
California	3,592	25,163	11,016	214.4	6,777	3,458	12.1	131,476	65,208	363.9	163,416	79,682	590.
Colorado	13,726	3,457	1,737	34.4	3,448	1,854	64.9	17,585	9,733	200.8	23,990	13,324	300.
Connecticut	15,772	537	268	1.7	2,745	1,364	2.7	11,006	5,294	10.7	14,288	6,926	15.
Delaware	3,997	1,051	550	15.4	2,302	1,150	4.0	7,298	3,987	29.5	10,691	5,687	48.
Florida	7,060	25,217	13,089	335.4	2,956	1,495	40.7	27,536	14,141	336.5	55,709	28,725	712.
Georgia	17,470	11,449	5,804	171.2	6,518	2,843	33.1	54,480	25,735	995.8	72,447	34,382	1,200.
Idaho	3,517	6,327	3,767	114.9	5,627	3,640	85.9	12,646	7,947	205.3	24,600	15,554	409.
Illinois	12,195	54,135	30,261	646.6	19,920	10,469	73.5	79,426	41,469	564.7	153,541	82,195	1,284.
Indiana	13,270	26,190	14,153	104.2	19,353	9,730	105.1	47,162	24,746	143.7	92,565	48,638	353.
Iowa	7,253	20,561	10,781	969.9	6,520	3,650	100.2	22,587	12,418	808.8	49,668	26,849	1,878.
Kansas	10,044	13,219	6,786	1,105.9	5,272	2,631	111.7	21,053	10,590	831.1	39,544	19,987	2,048.
Kentucky	10,998	12,909	6,904	344.6	4,101	2,134	22.8	31,323	15,736	323.5	48,403	24,774	690.
Louisiana	5,120	17,013	8,506	239.4	13,028	6,514	134.8	30,809	14,242	230.3	60,850	29,262	604.
Maine	3,934	8,570	4,570	55.4	2,483	1,315	9.8	15,155	7,687	128.8	26,208	13,572	194.
Maryland	5,780	23,722	12,565	72.6	3,885	1,837	8.2	11,182	5,084	101.2	38,789	20,286	186.
Massachusetts	15,418	8,099	4,039	30.5	1,306	903		49,372	23,098	45.2	59,277	28,040	75.
Michigan	12,357	45,773	23,785	654.6	11,764	5,941	140.1	41,535	20,954	340.9	99,072	50,680	1,135.
Minnesota	11,237	14,855	7,716	862.3	16,174	8,208	1,094.1	16,247	8,740	423.9	47,287	24,664	2,377.
Mississippi	5,252	16,561	8,036	527.6	5,723	3,025	152.3	23,000	11,820	515.9	45,724	22,881	1,195.
Missouri	12,657	17,353	8,986	1,027.6	7,732	4,321	41.3	71,801	37,138	1,163.6	96,946	50,445	2,232.
Montana	12,402	12,800	7,872	262.1	3,628	2,256	111.1	53,265	14,418	459.2	39,733	24,546	842.
Nebraska	8,605	26,731	15,066	1,092.5	4,486	2,248	91.3	23,602	13,260	628.9	56,839	30,576	1,812.
Nevada	9,896	1,700	1,421	77.2	2,626	2,322	42.7	6,801	5,557	86.3	11,127	9,300	206.
New Hampshire	4,496	3,217	1,608	21.7	1,141	732	1.3	6,658	3,252	40.3	11,016	5,592	63.
New Jersey	18,170	10,744	5,065	75.6	3,562	1,780	6.7	24,891	11,315	33.1	39,197	18,160	115.
New Mexico	5,517	4,287	2,598	123.2	2,156	1,371	33.0	13,098	8,316	232.0	19,541	12,385	394.
New York	35,434	50,091	26,254	126.3	24,351	11,829	75.8	202,022	94,748	243.8	276,464	132,831	445.
North Carolina	10,846	22,714	11,263	449.1	5,009	2,546	85.1	42,525	20,257	572.1	70,248	34,066	1,106.
North Dakota	8,734	14,744	7,390	1,646.2	10,174	5,259	578.6	5,604	2,804	311.8	30,522	15,453	2,536.
Ohio	14,439	52,290	26,221	115.8	9,104	4,842	35.2	61,775	28,941	112.5	123,169	60,004	263.
Oklahoma	20,015	5,854	3,164	189.5	9,008	4,577	125.0	24,350	12,883	337.9	39,212	20,634	648.
Oregon	7,047	7,133	4,236	105.8	1,130	694	16.5	13,144	8,136	169.8	21,407	13,066	292.
Pennsylvania	6,905	70,533	36,789	182.2	11,645	5,815	30.6	93,690	46,131	185.4	175,868	88,735	398.
Rhode Island	3,607	4,654	2,427	7.4	1,138	559	7.6	10,407	5,204	30.3	16,199	8,200	45.
South Carolina	11,433	8,246	4,502	201.2	1,675	916	9.0	16,811	8,861	285.5	26,732	14,279	495.
South Dakota	3,779	20,062	11,625	963.5	2,259	1,266	126.0	7,929	4,477	422.5	30,257	17,368	1,517.
Tennessee	12,409	17,759	8,333	373.7	10,109	5,054	80.7	34,945	15,554	509.5	62,813	29,441	969.
Texas	26,725	17,067	9,415	323.2	23,110	12,385	155.1	77,130	41,230	1,272.3	117,357	63,031	1,790.
Utah	1,548	8,457	4,386	158.4	3,618	2,632	50.2	6,543	5,013	105.0	18,618	14,034	313.
Vermont	3,913	3,190	1,665	32.4	854	439	2.4	7,457	3,714	64.2	10,500	5,218	100.
Virginia	15,535	10,807	5,369	205.9	3,798	1,885	19.7	20,818	10,260	295.1	35,483	17,514	520.
Washington	9,779	14,275	4,004	236.5	3,460	2,032	90.9	21,921	11,357	152.0	41,656	22,393	479.
West Virginia	11,716	10,695	5,427	67.2	3,351	1,698	35.4	14,352	7,190	50.8	28,398	14,315	152.
Wisconsin	9,461	26,908	13,114	478.8	10,521	5,271	116.8	30,342	15,449	319.6	67,771	33,834	915.
Wyoming	1,943	6,229	5,333	173.1	3,194	2,142	135.1	9,549	6,043	178.6	20,972	13,518	486.
Hawaii	3,760	1,722	861	2.5	2,004	1,002	.8	6,034	2,680	15.9	9,760	4,543	19.
District of Columbia	2,424	5,329	2,662	7.1	439	219	1.2	12,330	5,818	1.6	18,094	8,699	9.
Puerto Rico	2,406	7,878	3,871	35.6	1,801	863	9.0	12,278	5,430	40.6	21,957	10,164	85.
TOTAL	[illegible]	832,942	438,855	16,129.0	318,523	167,129	4,529.6	1,625,560	823,278	16,277.9	2,777,025	1,429,262	36,936.

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VOL. 28, NO. 9

AUGUST 1955

Public Roads

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OF COMMERCE,
WASHINGTON

In this issue: Passenger car performance on selected routes
(Shirley Memorial Highway and inset of U. S. 1 in Va.)

Public Roads

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Vol. 28, No. 9 *August 1955*

Published Bimonthly

Edgar A. Stromberg, *Editor*

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PUBLIC ROADS is sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1 per year (25 cents additional for foreign mailing) or 20 cents per single copy. Free distribution is limited to public officials actually engaged in planning or constructing highways, and to instructors of highway engineering. There are no vacancies in the free list at present.

The printing of this publication has been approved by the Director of the Bureau of the Budget, March 17, 1955.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

Contents of this publication may be reprinted. Mention of source is requested.

Operating Characteristics of a Passenger Car on Selected Routes

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by CARL C. SAAL
Chief, Vehicle Operations Section

To obtain data on the performance of typical automobiles under various highway operating conditions, a representative passenger car was driven some 28,000 miles on nine distinct studies during 1951 and 1952. The car was equipped with instruments to record the amount of time it operated at various speeds, rates of deceleration, percentages of maximum engine torque, and percentages of full throttle opening; the total fuel consumption and the amount of fuel used at various speeds; and the total trip time.

Five of the nine studies compared operations over a freeway having full control of access with grade separations and operations over a parallel major highway with intersections at grade and direct access to abutting property. The other studies were of a special nature made to evaluate the effect of traffic signals, sight distance, grade separation, and traffic congestion on the vehicle operational characteristics.

The data which were developed show some of the road-user benefits that may result through the use of a freeway instead of a parallel major highway, and establish basic relations between fuel consumption and highway gradient. The report also indicates the extent to which certain built-in vehicle characteristics are used in normal operation and discusses the relative advantages, in terms of fuel savings, of two methods commonly used to reduce gradients.

A KNOWLEDGE of certain operating characteristics of motor vehicles is essential in the development of standards and specifications for highways and for vehicles that will provide for the safe and efficient movement of traffic. In order to obtain data on the performance of typical passenger cars under varying highway operating conditions, the Committee on Vehicle Characteristics of the Highway Research Board, assisted by industry and government, developed instruments to record for any trip the amount of time that a vehicle operates at various speeds, rates of deceleration, percentages of maximum engine torque, and percentages of full throttle opening; the total fuel consumption and the amount of fuel used at various road speeds; and the total trip time.

The Bureau of Public Roads has made extensive use of these instruments to determine how these vehicle characteristics for a typical passenger car are related to various types of highway operations. A representative passenger car was operated some 28,000 miles on nine distinct studies during 1951 and 1952. Five of the nine studies dealt with operations over a freeway and over a parallel major highway. The other studies were of a special nature made to evaluate the effect of traffic signals, sight distance, grade separation, and traffic congestion on the vehicle operational characteristics.

This report is concerned essentially with the results of the studies which involved freeway operation. However, it covers briefly the studies of a special nature, and includes the results of special tests made to determine the fuel consumption and accelerating characteristics of the test vehicle on individual grades. The results reported here supplement those obtained by other investigators with the same set of instruments.

Although the basic data should have use in the fields of highway economics and design and within certain areas of automotive engineering, it is cautioned that the data represent only the performance of one 1951 model passenger car operated by the same driver throughout the tests. It may be farfetched to consider the performance data as representative of the average performance of passenger cars operating in the general traffic. On the other hand, it is believed that the performance of the test car on highway sections of varying geometric design may be compared to establish a relation which will be fairly representative of the relative performance of the average passenger car. Also, the relations established between fuel consumption, speed, and other variables may be reliably used to determine the relative advantages of various methods of reducing grades and estimating the fuel consumed on a given highway section.

Terminology

In order that there be a clear understanding of the discussions in this report, terms frequently used are here defined.

Freeway.—A divided arterial highway for through traffic with full control of access and with grade separations at intersections.

Major street or major highway.—Usage here is limited to arterial highways with intersections at grade and direct access to abutting property, and on which geometric design and traffic control measures are used to expedite the safe movement of through traffic.

Overall travel time.—The time of travel, including stops and delays except those off the traveled way.

Overall travel speed.—The speed over a specified section of highway, being the distance divided by overall travel time. The average for all traffic, or component thereof, is the summation of distance divided by the summation of overall travel times.

Composite performance.—The performance in given terms for a round trip over a specified section of highway. (Composite gasoline consumption in gallons per mile is the total number of gallons of gasoline required by a vehicle to travel in both directions on a section of highway, divided by twice the length of the section in miles.)

Directional performance.—The performance in given terms in a single direction over a specified section of highway.

Road-user benefits.—The advantages or savings that accrue to drivers or owners through the use of one highway facility as compared with the use of another. Benefits are measured in terms of the decrease in road-user costs and the increase in road-user services.

Total rise and fall.—The arithmetic sum of the vertical rise and fall in feet for any section of highway. (If the section of highway progressively rises 100 feet, falls 500 feet, rises 30 feet, and falls 10 feet, the total rise and fall will be 640 feet. The total rise and fall is the same regardless of the direction of travel.)

Rate of rise and fall.—The total rise and fall for any section of highway divided by the length of section in hundreds of feet. (It is

not to be confused with the percent of grade. It is equivalent to the average percent of grade only when either the rise or fall is 100 percent of the total rise and fall.)

Average test method.—The driver travels at a speed which in his opinion is representative of the speed of all traffic at the time, without trying to keep a balance in the number of passings.

Attempted speed test method.—The driver attempts to maintain a specified speed over a section of highway, passing all vehicles that interfere with maintaining the specified speed, and exceeding the specified speed only during the passings.

Maximum torque.—The maximum engine torque at a specified engine speed or corresponding road speed.

Purpose of Report

The specific purposes of this report are as follows: To show some of the road-user benefits that may result through the use of a freeway instead of a parallel major highway, to determine the extent to which certain built-in vehicle characteristics are used in normal operation, to establish basic relations between fuel consumption and highway gradient, and between acceleration and highway gradient, to evaluate several methods used to estimate the fuel consumed on a highway section, and to determine the relative advantages, in terms of fuel savings, of two methods commonly used to reduce gradients.

Summary of Findings

The pertinent findings described here refer specifically to the operations of the test passenger car. Definite conclusions as to the overall performance of passenger cars in the general traffic cannot be formed from the results of tests on a single passenger car operated by the same driver on all tests. Only indications of the overall performance of passenger cars should be read into any of the findings.

1. For each of the five freeway studies, considering the total lengths, the test car would have had to travel over the freeway at a slower speed than the average overall travel speed reported for all passenger cars using the facility, in order to realize the same rate of fuel consumption as observed on the parallel major highway. Therefore, if the test car were to maintain prevailing overall travel speeds on comparable roads, the consumption per mile would be higher on each freeway than on the parallel major highway.

2. Unless a major highway has a much greater rate of rise and fall or is much more congested than a parallel freeway, the latter will show a higher rate of consumption when the vehicle is operated at the average overall travel speeds found on the two roads. For example, the consumption per mile at the prevailing average overall travel speeds was lower on the western extension of the Pennsylvania Turnpike than on the highly urbanized section of the parallel route extending through Wilkinsburg and Pittsburgh, Pa.

3. A sizable time savings resulted in each case from the use of a freeway, instead of a major highway, at the average overall travel speeds found on the two roads.

4. Except in one case, the use of the freeway instead of the parallel major highway saved enough travel mileage to make the fuel consumption in gallons approximately the same for a composite trip over either facility when the vehicle was operated at the average overall travel speeds found on the two roads.

5. Where the average overall travel speed on a freeway was below 40 miles per hour, the use of such facility instead of a major highway showed sizable savings in gasoline during the peak traffic periods.

6. The percentage of time spent in braking was nearly zero on a freeway and very small on a major highway; however, the time spent in braking on a major highway was as much as 34 times greater than that spent on a freeway. The maximum rate of deceleration recorded on any test was about 60 percent of the potential rate of deceleration built into the car.

7. Maximum engine torque and full throttle opening were used only a very small portion of the time on either a freeway or a major highway. Less than half of the potential torque and power were normally utilized on any test run. The average engine torque and throttle opening observed on a major highway were appreciably less than that observed on the parallel freeway at the average overall travel speeds found on the two roads.

8. The relations established between fuel consumption and rate of grade, and between fuel consumption and rate of rise and fall were very similar. In general, the rate of consumption increases about in direct proportion to the increase in grade or rate of rise and fall up to 6 percent. Above 6 percent, the increase is at a faster rate.

9. A reduction of grades exceeding 6 percent resulted in appreciable savings in fuel consumption, whether or not the reduction involved a decrease in rise and fall. However, reduction of grades between 4 and 6 percent produced no substantial savings unless the grade reduction also reduced rise and fall. A reduction of 3- and 4-percent grades did not result in an appreciable savings, even if rise and fall was also reduced.

10. The use of the rate of total rise and fall of a section of highway to estimate fuel consumption on the section was found to be as accurate as a more complicated method that involved the consideration of each individual grade.

Scope of Studies

Freeway studies

In selecting the five pairs of test routes for studying some of the road-user benefits that might result from the use of freeways by passenger cars, an effort was made to cover as wide a range of highway conditions as possible in the Eastern United States. The five freeways selected for study were the New Jersey Turnpike, the middle section of the Pennsylvania Turnpike, the Maine Turnpike, the western section of the Pennsylvania Turnpike, and the Shirley Memorial Highway in Virginia. Only the latter route was free of toll. The parallel major highway in each instance was the alternate route that would be commonly used to travel between the same termini.

Figures 1 through 5 show sketches of the general layout of the test routes for each study and the profiles for each pair of routes, except for the Maine Turnpike study. These profiles were plotted from elevations measured with an altimeter. Each of the routes, except the western section of the Pennsylvania Turnpike, was divided into test sections by control points located at definite changes in the character of the profile or traffic flow. The operating characteristics of the test vehicle, within each section, were recorded at these control points.

All of the freeways were built approximately to the same design standards. The maximum grade was not over 3 percent in any case, and the rate of rise and fall varied from 0.8 for the New Jersey Turnpike to 1.4 for the two sections of the Pennsylvania Turnpike. It could be expected that the test car would perform about the same on each of the five freeways.

In contrast each route paralleling a freeway afforded a conglomeration of surface types, pavement widths, curvatures, and gradients. There was also considerable variation in the design characteristics between the various parallel routes. The rates of rise and fall varied from 0.9 for the route paralleling the New Jersey Turnpike to 3.3 for the route paralleling the middle Pennsylvania Turnpike. The parallel major highway and the turnpike had approximately the same rate of rise and fall in the case of the New Jersey and Maine studies. The rates of rise and fall for the routes paralleling the middle and western sections of the Pennsylvania Turnpike, and the Shirley Memorial Highway were about 2.4, 1.4, and 1.3 times that for the respective freeway. In addition to the wide range in the character of the profiles, the routes paralleling the freeways differed materially from each other in other ways which had a bearing on the results obtained. This can best be brought out by a brief description of each parallel major highway.

Generally, the parallel major highway in New Jersey was of four-lane construction with fair alinement except for the southern section between control points 1 and 2 (see fig. 1). This southern section was essentially of two-lane construction with poor alinement. The test car encountered traffic congestion particularly on section 1–2; within the numerous small municipalities that lie on the route from control points 1 to 6; on the bypass around Camden in section 2–3; and on parts of the sections between control points 6 and 10 where the route passed through a highly urbanized area. The congestion was most severe from control points 8 to 10, which extend from the eastern approach of the Pulaski Skyway to the George Washington Bridge.

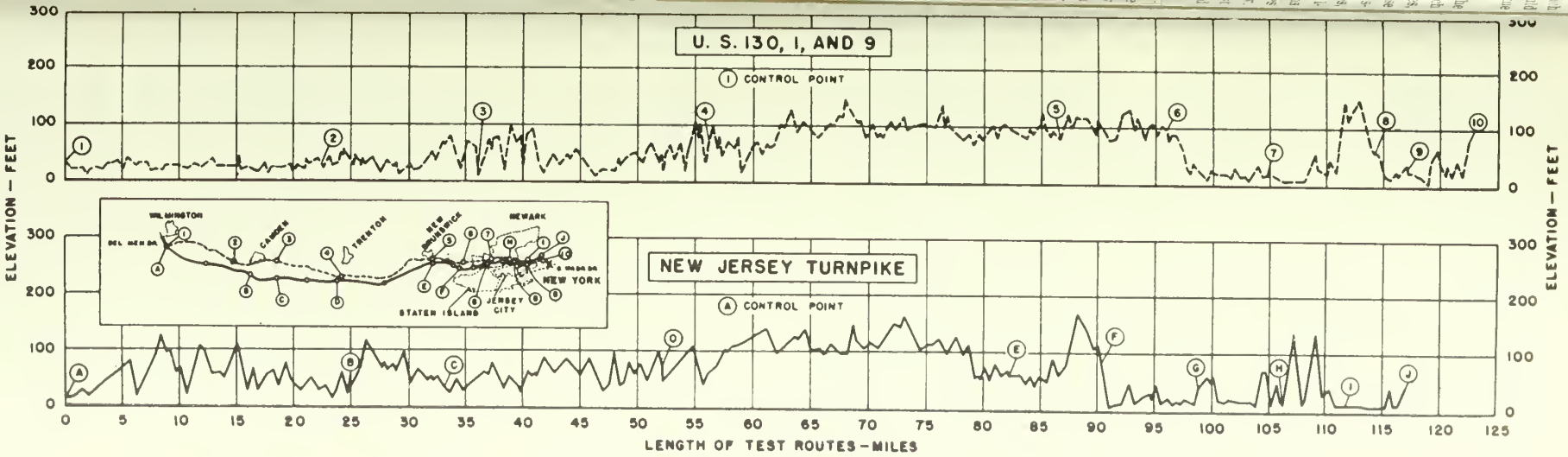

Figure 1.—Profile and sketch of New Jersey Turnpike and U. S. 130, 1, and 9 between the Delaware Memorial Bridge and the George Washington Bridge.

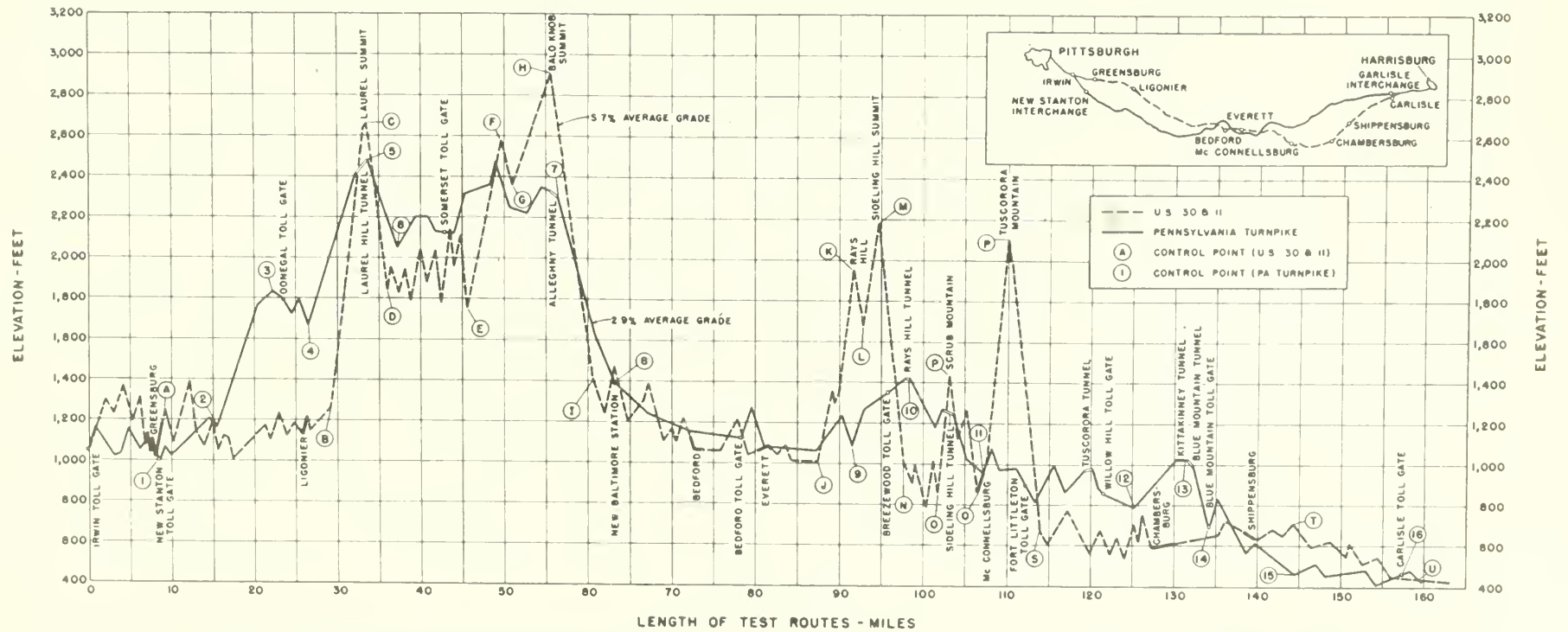

Figure 2.—Profile and sketch of the Pennsylvania Turnpike and U. S. 30 and 11 between Carlisle and Irwin interchanges.

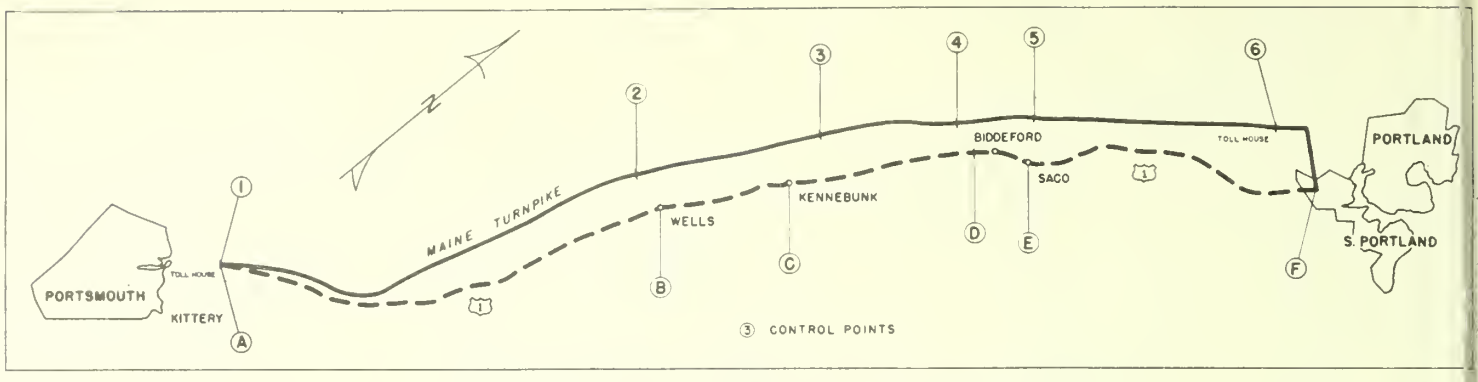

Figure 3.—Sketch of the Maine Turnpike and U. S. 1.

In Maine, the parallel route was a two-lane highway with rather poor alinement except for a short section near Portland. The test car was frequently slowed by passage through municipalities varying in population from a few hundred to over 20,000.

The route paralleling the middle section of the Pennsylvania Turnpike generally consisted of two lanes varying in individual width from 9 to 12 feet. Only a small mileage had lanes wider than 10 feet. Narrow shoulders, sharp curves, and restricted sight distances were the rule. The greater portion of the route had a bituminous surface with high crown prevailing in many sections. The operation over this route may be classed as strictly rural, since there are only six towns of any size, the largest of which was about 17,000 population. Traffic congestion was only a minor factor in the test runs on this route. The important factors with respect to passenger car operations were gradient and poor alinement.

The western portion of the Pennsylvania Turnpike bypasses Wilkinsburg, Pittsburgh, and an almost continuous string of municipalities which dot the north bank of the Ohio River between Pittsburgh and Rochester. The parallel major highway was principally urban for about 70 percent of its length.

In Virginia, U. S. 1 parallels the Shirley Memorial Highway and passes through Alexandria and its environs which constitute over 30 percent of the test route. Restricted speed zones also exist through areas of heavy roadside development and through a military reservation. Actually more than 50 percent of the route is zoned for a maximum speed of 35 miles per hour or less. This route in the rural areas is a four-lane highway with fair alinement.

Special studies

One of the four special studies was made to supplement data previously obtained by tests of vehicle performance on an old road and subsequently on a complete relocation of improved alinement between a junction near Frederick and the city limits of Hagerstown, Md. The sketch and profiles of the two test routes are shown in figure 6. In length and rise and fall, there is little to choose between the two locations. The rates of rise and fall were 3.7 for the new road and 4.1 for the old road, the highest rates of all the test routes. Moreover, on each road grades range as steep as 8 percent, and on each, heavy grades run a mile or more in length. The big difference between the two roads lies in the percentage of the total lengths that permit passing. On the old road 49.3 percent in one direction and 45.6 percent in the other, or nearly half of the total length, was marked for no passing. On the new road only 12.2 percent of the length in one direction and 11.6 percent in the other would not permit safe passing.

Another special study involved two possible routes between two bridges across the Potomac River at Washington, D. C., and Annandale, Va. (see fig. 7). This study was made primarily to obtain average running times of passenger cars for use in a study of the effect of travel time and distance on freeway usage.[1] However, while the running times were being observed the other vehicle characteristics were also studied. The first leg of each route was identical, being a rather low-speed freeway operation (posted limit of 40 miles per hour) on the Pentagon network. One of the routes followed Columbia Pike to Annandale, on which there were numerous intersections at grade, and on which there was heavy traffic congestion during the morning and evening peaks. The other route, included a section of the Shirley Memorial Highway and Virginia State Route 236. About two-thirds of the latter route was a freeway as compared to about one-fourth of the route to Annandale by way of Columbia Pike.

A third study was made for the Regional Highway Planning Committee for Metropolitan Washington to aid in determining the need for constructing an interchange ramp at 14th Street, SW., and Maine Avenue in Washington, D. C., which would eliminate an at-grade intersection for traffic desiring to make a left turn from Maine Avenue into 14th Street. A grade separation had been built at this location, but the one intersection leg was retained at grade because the ramp had to pass through a corner of the Bureau of Engraving and Printing Building. Only travel time and fuel consumption were measured on this study during both the peak and off-peak traffic periods.

The fourth special study was made on a 2-mile section of Columbia Pike between 4-Mile Run Drive and Scott Street as indicated in figure 7. Tests were made during peak a off-peak periods when there were 2 traffic li installations, and then repeated when additional traffic-actuated signals had b installed within the same section.

Special tests

In addition to the freeway and spec studies that have just been described, te were made to determine the fuel consumpti and accelerating ability of the test car individual grades of 0.0, 2.84, 6.0, and 8.0 p cent. The grades were 1.00, 0.40, 0.284 a 0.50 miles in length, respectively. All of th grades were at elevations of 900 feet or le and all except the 8.0-percent grade were s faced with portland cement concrete. T 8-percent grade was paved with a high-ty bituminous concrete.

Test Procedure

Freeway and special studies

The instruments installed in the test c were described in detail in a previous repo For that reason, this report will consider on the type of information collected and t procedures employed.

A typical field data sheet is shown in f ure 8 for the southernmost section of t major highway paralleling the New Jers Turnpike. The recording apparatus co sisted of five banks of 10 counters each. electric clock, and a master time count These counters were actually arranged in t same pattern as the field data sheet. Ea count represented one second on the banks counters for speed, braking, engine torqu and throttle opening; and one-thousandth a gallon on the bank of counters for gasoli consumption. Each counter of a bank repr sented a class interval of the particular ite being studied. The units of the class inte vals were miles per hour for speed and gas line consumption, feet per second per seco for braking, and percent for engine torque a throttle opening. The range in the cla intervals for each bank of counters is show in figure 8.

The time read from the electric clock w used to check the proper functioning of th master counter, and in turn, the time ind

[1] *The effect of travel time and distance on freeway usage*, by Darel L. Trueblood. PUBLIC ROADS, vol. 26, No. 12. Feb. 1952.

[2] *A study of vehicle, roadway, and traffic relatio ships by means of statistical instruments* by Thom J. Carmichael and Charles E. Haley. Proceedin of the Highway Research Board, vol. 30, 1950, p 282–296.

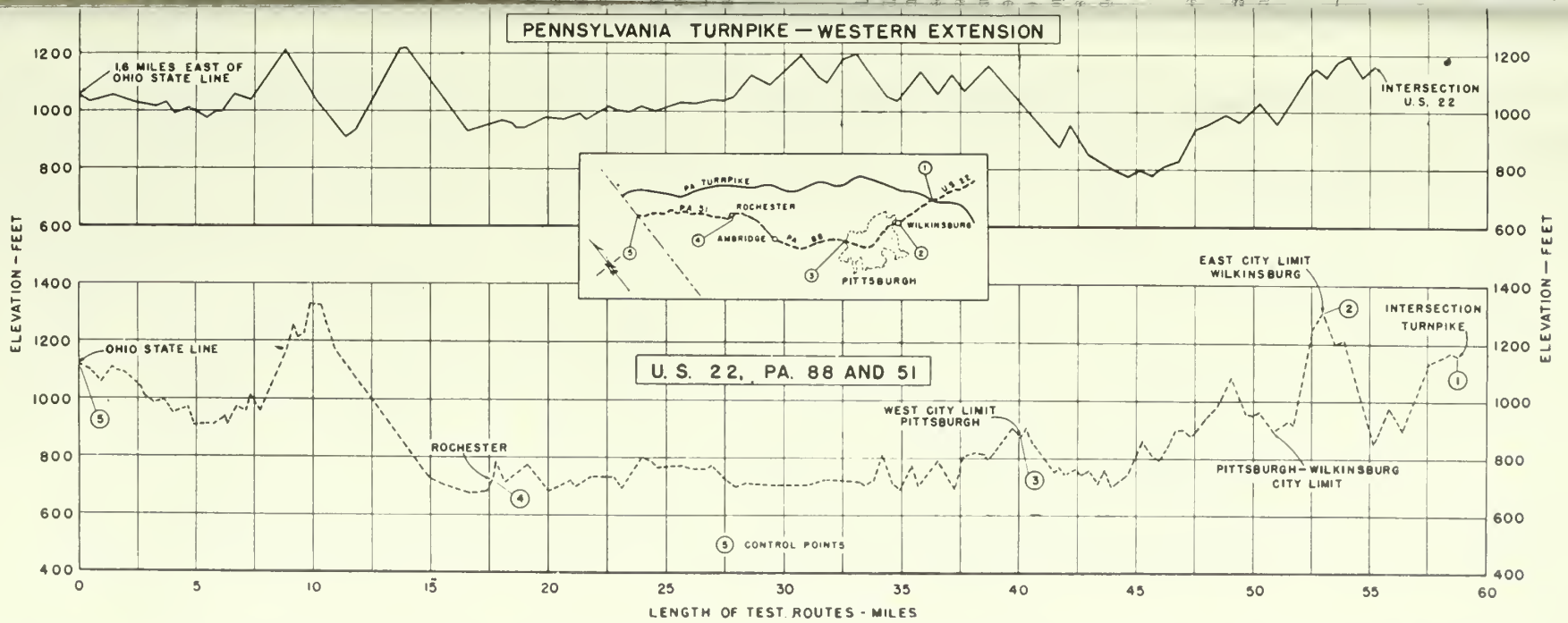

Figure 4.—Profile and sketch of the western extension of the Pennsylvania Turnpike and U. S. 22, Pa. routes 88 and 51 between the Ohio-Pennsylvania line and junction of U. S. 22 and the Turnpike east of Pittsburgh.

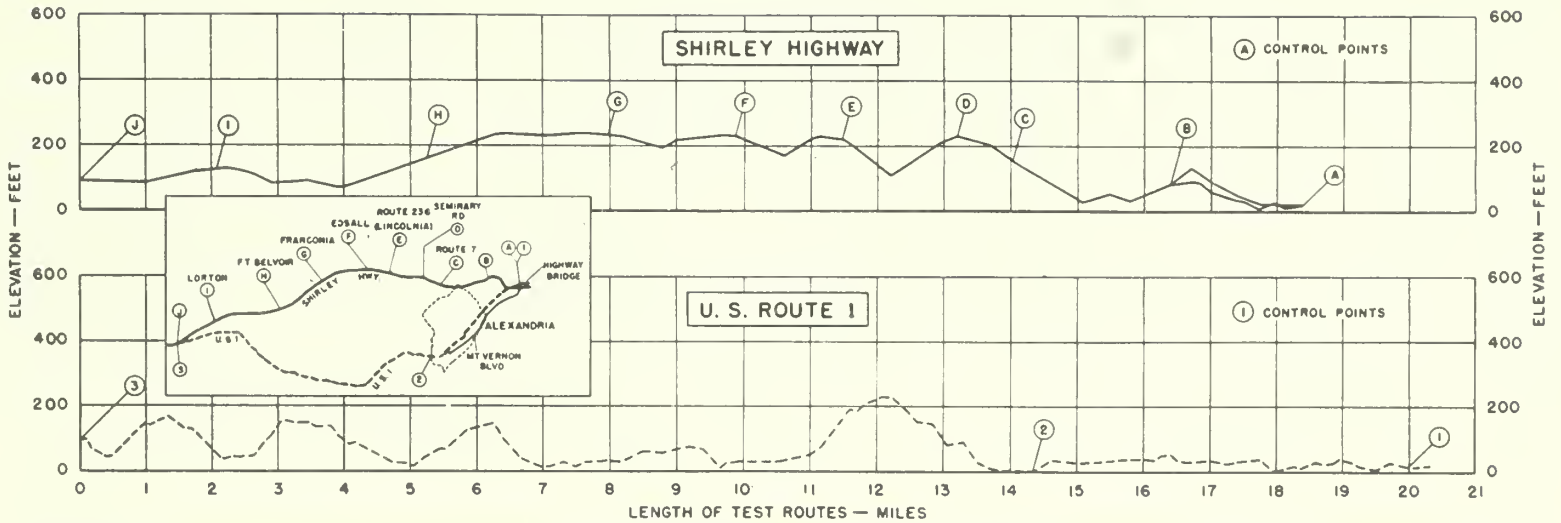

Figure 5.—Profile and sketch of the Shirley Memorial Highway and U. S. 1 between Highway Bridge and junction near Woodbridge, Va.

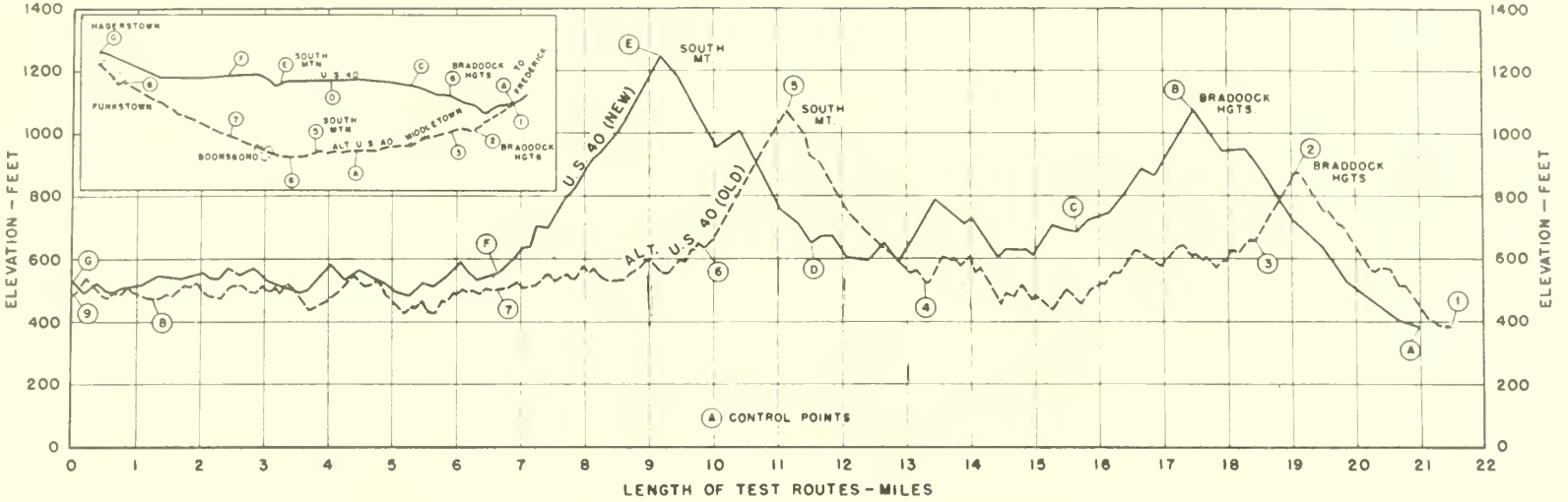

Figure 6.—Profile and sketch of U. S. 40 and alternate U. S. 40 between Hagerstown and junction near Frederick, Md.

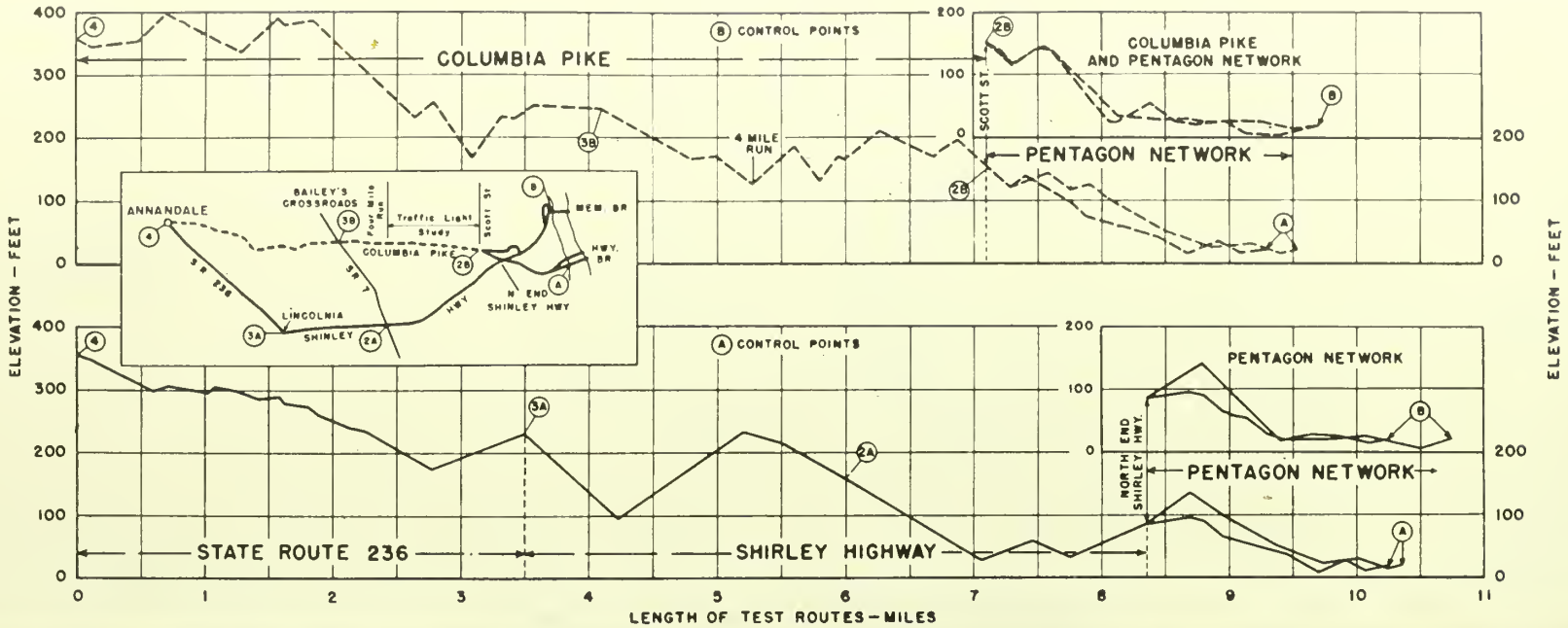

Figure 7.—Profile and sketch of test routes between Annandale, Va., and Highway and Memorial Bridges, Washington, D. C.

cated by the master counter was used to ascertain that all counters of a given bank were functioning properly. It is seen that the total time counts shown opposite the counter banks checked very closely with the master time counter. Likewise, the trip time from the electric clock compares closely with that of the master counter. As indicated in figure 8, the end results were an average rate of speed and gasoline consumption, percentage of time spent in each range of speed, deceleration, percentage of maximum torque and full throttle opening, and percentage of gasoline used in the various speed ranges. The time recorded on the master time counter was used to compute the average speed.

Engine torque was not directly recorded, instead it was assumed to be proportional to the pressure existing in the intake manifold. The intake manifold vacuum instrument consisted of a metal bellows to which was attached a calibrated spring and a swing arm that passed over a sector divided into contact segments representing ranges in vacuum. These ranges in vacuum were assigned engine torque values in percentage of maximum torque, as shown in figure 8. The maximum torque referred to in this instance roughly approximates the maximum for the engine speed or corresponding road speed at the instant of recording. It is not to be confused with the peak engine torque. The percentage values can be converted roughly to pound-feet of torque or pounds of tractive effort by assuming an average maximum torque for the entire range of engine speed involved.

The average test method was used when the traffic volume was dense enough for the

Test No. B-Run 2 Date October 23, 1951

Route U. S. 130, 1 and 9, Major highway parallel to New Jersey Turnpike

Remarks Section 1 - 2, Northbound
Deepwater to Thorofare, U. S. 130 before opening of Turnpike

SPEED

Counter No.	1	2	3	4	5	6	7	8	9	10	Total counts
Speed, MPH	0-5	6-8	9-11	12-18	19-23	24-35	36-46	47-56	57-68	69 & up	
Finish	36349	43778	22724	57671	63082	36613	32259	18835	46931	07078	
Start	36335	43777	22720	57650	63031	35894	31306	18487	46931	07078	
Difference	14	1	4	21	51	719	952	348	0	0	2110
% Total	0.7	1/	0.2	1.0	2.4	34.1	45.1	16.5	0	0	

GASOLINE CONSUMPTION

Counter No.	1	2	3	4	5	6	7	8	9	10	Total counts
Speed, MPH	0-5	6-8	9-11	12-18	19-23	24-35	36-46	47-56	57-68	69 & up	
Finish	33046	49290	59817	92727	60401	40136	99300	00050	51920	03729	
Start	33045	49290	59816	92723	60384	39833	98752	99769	51920	03729	
Difference	1	0	1	4	17	303	548	281	0	0	1155
% Total	0.1	0	0.1	0.3	1.5	26.2	47.5	24.3	0	0	

BRAKING

Counter No.	1	2	3	4	5	6	7	8	9	10	Total counts
Deceleration	0-3	4-7	8-10	11-13	14-16	17-19	20-23	24-26	27-29	30 & up	
Finish	74418	29629	06203	02380	02206	02542	02141	06856	85677	03158	
Start	72309	29614	06203	02380	02206	02542	02141	06856	85677	03158	
Difference	2109	15	0	0	0	0	0	0	0	0	2124
% Total	99.3	0.7	0	0	0	0	0	0	0	0	

ENGINE TORQUE

Counter No.	1	2	3	4	5	6	7	8	9	10	Total counts
% Torque	Coast	0-10	11-21	22-32	33-43	44-55	56-66	67-77	78-88	89-100	
Finish	77642	79928	19508	14947	27422	41757	25848	21789	43770	08527	
Start	77588	79679	19103	14367	26790	41630	25776	21741	43751	08526	
Difference	54	249	405	580	632	127	72	48	19	1	2187
% Total	2.5	11.4	18.5	26.5	28.9	5.8	3.3	2.2	0.9	1/	

THROTTLE OPENING

Counter No.	1	2	3	4	5	6	7	8	9	10	Total counts
% Opening	0-9	10-19	20-29	30-39	40-49	50-59	60-69	70-79	80-89	90-100	
Finish	44203	84569	82488	98040	41216	31988	26359	23185	29305	46465	
Start	44164	83742	81669	97610	41189	31988	26359	23185	29305	46465	
Difference	39	827	819	430	27	0	0	0	0	0	2142
% Total	1.8	38.6	38.2	20.1	1.3	0	0	0	0	0	

ODOMETER		CLOCK		MASTER COUNTER	
Finish 8367.45		Finish 5:52 P.M.		Finish 15014	
Start 8345.20		Start 5:17 P.M.		Start 12891	
Trip Mileage	22.25	Trip Time (Seconds)	2100	Total Counts (Seconds)	2123

Trip Average Speed 37.7 MPH

Trip Average Gas Consumption 19.3 MPG

1/ Less than 0.05%

Figure 8.—A typical field data sheet used in recording information on vehicle operating characteristics.

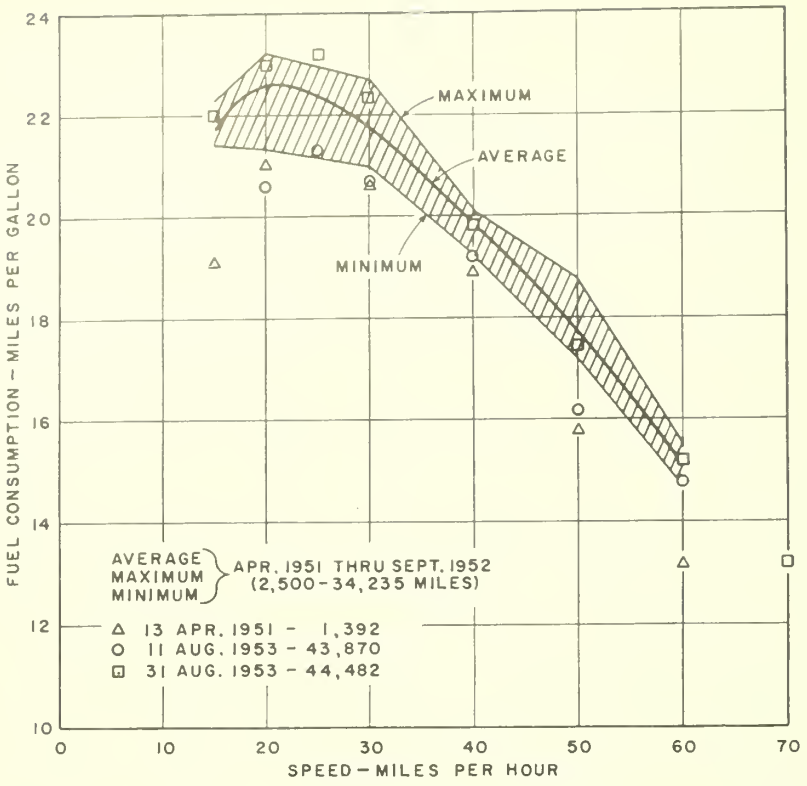

Figure 9.—Fuel calibration of test car in third gear with burette on a 1-mile level section for various sustained speeds.

Table 1.—Horsepower and torque data of test car [1]

Road speed in third gear	Maximum gross horsepower	Maximum gross torque
M. p. h.		*Lb./ft.*
20	34	185
25	44	191
30	54	191
35	63	189
40	72	186
50	85	178
60	94	163
70	96	143
80	91	119

[1] Taken from curves in Manufacturer's Shop Manual.

Memorial Highway. Test runs were made in both directions over the section at speeds of 15, 20, 25, 30, 40, 50, and 60 miles per hour.

The results of 13 such calibration tests are shown in figure 9. The average consumption rates in miles per gallon, between April 1951 and September 1952 when the odometer readings ranged from 2,500 to 34,235 miles, are shown by the smooth curve. The variation in rates of consumption from the average during this period is indicated by the maximum and minimum values, each of which is connected by a series of straight lines. The percentage of variation from the average ranged from 1.4 to 6.2 percent. In view of this rather small variation, which was

driver to reliably approximate the speed of all traffic at a given instant. Where the average test method was not feasible, test runs were made on a particular section at three or more attempted speeds so that the rate of fuel consumption could be interpolated for an average overall travel speed of all passenger cars obtained from other sources. Attempted speeds greater than 60 miles per hour were not possible, because the fuelmeter did not have sufficient volume to supply the flow of fuel required to negotiate existing grades at higher speeds.

Three test runs were made over each test route in each direction at each attempted speed for all except two of the studies. For the intersection study at Maine Avenue and 14th Street, Washington, D. C., 12 test runs were made in the off-peak period and 26 test runs in the peak period. For the traffic light study on Columbia Pike (see fig. 7), 4 and 16 test runs were made before the installation of additional traffic lights during the off-peak and peak periods, respectively. After the installation, 6 and 18 test runs were made during the off-peak and peak periods, respectively. The test runs were scheduled so that a particular test section or route would be traveled at different times during the period of study.

Fuel calibration

In order to maintain the fuel characteristics of the test car at approximately the same level throughout the period of the study, calibration tests were conducted before and after most of the studies. The fuel consumption of the test car was checked with a burette on a measured mile located on the Shirley

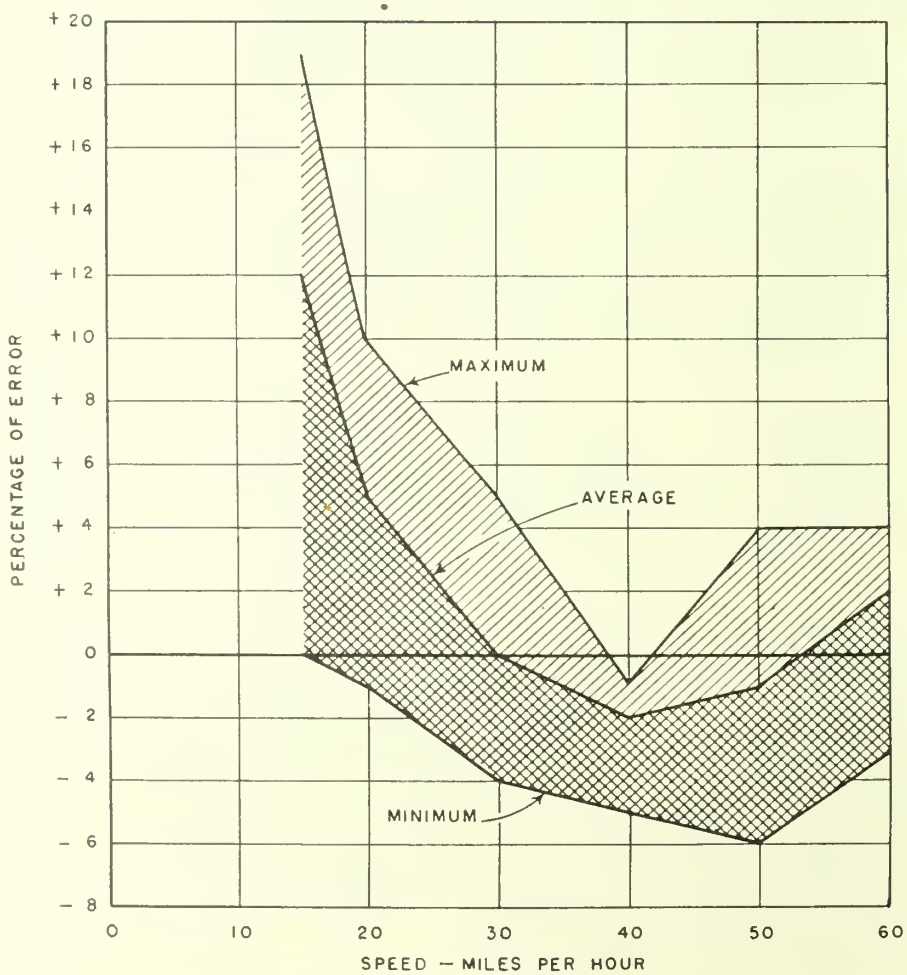

Figure 10.—Calibration of fuelmeter with a burette on a 1-mile level section for various sustained speeds during period, Apr. 1951–52.

Table 2.—Summary of average composite performance of test vehicle on various routes

Type of facility [1]	Test route	Date of tests	Period of study [2]	Length of route	Rise and fall	Speed		Fuel consumption (corrected)	Braking				Average engine torque	Average throttle opening
						Attempted	Average		Percent time 0–3 ft./sec.[2]	Percent time Over 3 ft./sec.[2]	Maximum deceleration	Time factor		
				Miles	*Ft./100 ft.*	*M. p. h.*	*M. p. h.*	*M. p. g.*	*Percent*	*Percent*	*Ft./sec.²*	*Sec./100 mi.*	*Percent*	*Percent*
1A Freeway	Delaware Memorial Bridge to George Washington Bridge via New Jersey Turnpike.	Apr. 1952	8 a. m.–6 p. m	116.3	0.8	40	39.4	18.6	100.0	[3]	8–10	2.6	29.0	17.1
						50	48.6	17.2	100.0	[3]	8–10	2.4	33.8	21.1
						60	58.1	15.4	99.9	0.1	11–13	5.3	45.4	34.1
1B Major highway.	Delaware Memorial Bridge to George Washington Bridge via U. S. 130, 1, and 9.	Oct. 1951	do	122.2	.9	[4]	38.3	17.4	98.1	1.9	14–16	181.2	31.4	25.7
		Apr. 1952	do	-----	----	[4]	40.7	17.2	98.2	1.8	14–16	159.0	34.8	20.3
2A Freeway	Carlisle interchange to New Stanton interchange via Pennsylvania Turnpike.	[5]	do	148.7	1.4	40	40.2	18.8	100.0	[3]	11–13	2.7	27.0	14.7
		[5]	do	-----	----	50	49.0	16.8	99.9	.1	8–10	7.6	33.5	17.8
		June 1952	do	-----	----	60	57.1	15.1	99.7	.3	11–13	18.5	42.6	31.3
2B Major highway.	Carlisle to Greensburg, Pa., via U. S. 11 and 30 (including larger towns).	do	do	149.4	3.3	30	30.6	17.6	99.4	.6	11–13	70.5	30.2	-----
						40	38.0	16.6	99.0	1.0	11–13	93.5	32.6	-----
						50	42.7	15.6	97.6	2.4	14–16	196.8	36.3	-----
2C Major highway.	Carlisle to Greensburg, Pa., via U. S. 11 and 30 (excluding larger towns).	do	do	140.3	3.4	30	31.6	17.5	----	----	----	----	-----	-----
						40	40.3	16.5	----	--	-- -	----	-----	-----
						50	46.0	15.5	----	----	-- -	---	-----	-----
3A Freeway	Kittery to Portland, Maine, via Maine Turnpike.	Aug. 1952	do	41.8	1.2	40	39.8	19.3	----	-	---	--	---	-----
						50	49.0	16.5	----	----	----	---	-----	-- --
						60	58.8	14.9	----	----	----	----	-----	- ---
3B Major highway.	Kittery to Portland, Maine, via U. S. 1.	do	Weekday	43.8	1.3	[4]	36.4	17.9	----	---	----	--	-----	- --
		do	Weekend	-----	----	[4]	35.1	17.7	----	----	--	- -	-----	-- -
4A Freeway	Pittsburgh interchange to Ohio State line via Pennsylvania Turnpike.	July 1952	8 a. m.–6 p. m.	55.2	1.4	40	40.3	19.0	----	----	--	----	--- -	--
						50	49.8	17.4	----	----	----	----	-----	-----
						60	58.8	15.7	----	----	----	-- -	-----	-----
		Oct. 1952	do	-----	----	40	39.9	19.1	----	---	----	----	----	- ---
						50	49.9	17.1	----	----	-- -	- -	-----	-
						60	58.8	15.6	----	----	-- -	----	-----	--- -
4B Major highway.	Pittsburgh interchange to Ohio State line via U. S. 22, Pa. 19 (alternate), 88, and 51 (through Pittsburgh).	Dec. 1951	do	58.5	2.0	[4]	26.4	16.7	----	----	----	----	-----	- ---
		do	do	[6] (17.6)	1.9	[4]	35.8	18.2	----	----	----	----	-----	-----
		do	do	[7] (40.9)	2.1	[4]	23.8	16.2	----	----	----	----	-----	--
		July 1952	do	[7] 40.9	2.1	[4]	25.9	16.7	-- -	----	--	- --	- --	--
		Oct. 1952	do	[7] 40.9	2.1	[4]	25.1	16.6	----	----	-- -	-	-----	-----
		July 1952	do	[8] 12.9	2.7	[4]	18.3	14.9	----	----	--	-	- -	-----
5A Freeway	Washington, D. C., (Highway Bridge) to Woodbridge, Va., via Shirley Memorial Highway.	Dec. 1951	Off-peak	[9] 18.4	1.3	[4]	49.8	17.9	99.7	.3	8–10	19.7	38.8	24.7
		Mar. 1954	do	-----	----	[4]	50.9	17.2	----	----	----	- -	-----	-----
		do	do	14.1	1.1	30	30.8	21.1	- -	----	----	----	-----	--- -
		do	do	-----	----	40	40.6	19.6	----	----	----	----	-----	-----
		do	do	-----	----	50	49.5	17.9	----	----	----	----	-----	-- --
		do	do	-----	----	55	53.2	16.8	----	----	----	----	-----	-----
5B Major highway.	Washington, D. C., (Highway Bridge) to Woodbridge, Va., via U. S. 1.	Dec. 1951	do	20.3	1.7	[4]	33.8	18.9	98.9	1.1	11–13	120.7	31.0	17.7
		do	do	[10] (6.0)	1.0	[4]	23.6	18.1	97.9	2.1	11–13	318.8	28.7	12.4
		do	do	[6] (14.3)	1.9	[4]	40.7	19.2	99.5	.5	8–10	43.4	32.7	21.2
5C Major highway.	Washington, D. C., (Highway Bridge) to Woodbridge, Va., via Mount Vernon Blvd. and U. S. 1.	do	do	20.4	1.7	[4]	36.4	18.8	98.7	1.3	11–13	127.0	31.8	20.6
		do	do	[10] (6.1)	1.0	[4]	28.8	17.7	97.3	2.7	11–13	335.8	30.3	17.1
		do	do	[6] (14.3)	1.9	[4]	40.7	19.2	99.5	.5	8–10	43.4	32.7	21.2
6A Major highway.	Frederick to Hagerstown, Md., via new U. S. 40.	July 1951	8 a. m.–6 p. m.	21.0	3.7	30	32.3	18.5	100.0	[3]	4–7	1.2	26.8	23.1
		do	do	-----	----	40	40.9	17.5	100.0	0	0–3	0	30.3	26.2
		Sept. 1952	do	-----	----	40	39.6	17.5	99.1	.1	4–7	7.2	-----	-----
		July 1951	do	-----	----	50	49.4	16.2	99.7	.3	27–29	22.8	34.2	29.4
		Aug. 1952	do	-----	----	50	48.5	15.8	----	----	----	- -	-----	-----
		Sept. 1952	do	-----	----	50	47.7	16.0	99.7	.3	8–10	21.6	-----	--
		July 1951	do	-----	----	60	53.4	14.8	99.6	.4	8–10	28.7	41.0	33.9
		Sept. 1952	do	-----	----	60	54.6	14.6	98.8	1.2	8–10	79.1	-----	-----
6B Major highway.	Frederick to Hagerstown, Md., via old U. S. 40.	July 1951	do	21.5	4.1	[11]	35.9	16.6	99.2	.8	8–10	82.4	29.9	20.7
7A Freeway	Washington, D. C., (Highway Bridge) to Annandale, Va., via Shirley Memorial Highway.	do	Peak	10.3	1.8	[4]	40.0	16.4	98.6	1.4	8–10	120.8	31.4	26.2
		do	Off-peak	-----	----	[4]	43.9	17.7	99.5	.5	4–7	38.7	33.8	28.8
7B Major highway.	Washington, D. C., (Highway Bridge) to Annandale, Va., via Columbia Pike.	do	Peak	9.4	2.4	[4]	26.6	15.4	97.5	2.5	8–10	315.3	29.3	20.1
		do	Off-peak	-----	----	[4]	33.1	17.8	98.2	1.8	8–10	219.1	28.5	20.4
7C Freeway	Washington, D. C. (Memorial Bridge), to Annandale, Va., via Shirley Memorial Highway	do	Peak	10.5	1.8	[4]	41.0	16.4	99.7	.3	4–7	28.5	29.9	26.2
		do	Off-peak	-----	----	[4]	45.3	17.7	98.6	1.4	8–10	114.0	35.1	29.6
7D Major highway.	Washington, D. C., (Memorial Bridge) to Annandale, Va., via Columbia Pike	do	Peak	9.7	2.4	[4]	26.5	15.4	97.3	2.7	11–13	366.9	30.0	19.5
		do	Off-peak	-----	----	[4]	34.3	17.4	98.6	1.4	11–13	149.9	29.1	22.5
8 Major street	Washington, D. C., (1301 Maine Ave.) to Inlet Bridge	Oct. 1951	Peak	0.23	0.2	[4]	8.9	9.6	----	-- -	- --	---	---	--- -
		do	Off-peak	-----	----	[4]	18.4	13.1	----	----	----	----	-----	- ---
9 Major highway	Arlington, Va., (Columbia Pike from 4-mile Run Drive to Washington Blvd. underpass)	Apr. 1952	Peak	2.00	3.1	[4]	21.4	13.3	----	-- -	- -	--- -	---	--
		do	Off-peak	-----	----	[4]	26.1	16.3	----	----	-- -	--	---	
		Aug. 1952	Peak	-----	----	[4]	21.5	14.2	---	---	-- --	--	---	-
		do	Off-peak	-----	----	[4]	24.9	14.6	- -	-	-	-	---	-

[1] For comparison, facilities are numbered to indicate freeways and major parallel highways.
[2] A minimum of 3 round trips was made over each test route spaced to cover the period indicated.
[3] Less than 0.05 percent.
[4] Average test method used.
[5] 1 test run in December 1951 and 2 test runs in June 1952.
[6] Rural traffic conditions.
[7] Urban traffic conditions.
[8] Through Wilkinsburg and Pittsburgh, Pa.
[9] Speed limit posted at 40 m. p. h. for 1.9 miles, 50 m. p. h. for 2.4 miles, and 55 m. p. h. for 14.1 miles.
[10] Through Alexandria, Va.
[11] Attempted to drive average speed (33.6 m. p. h.) for passenger cars observed before opening of new U. S. 40.

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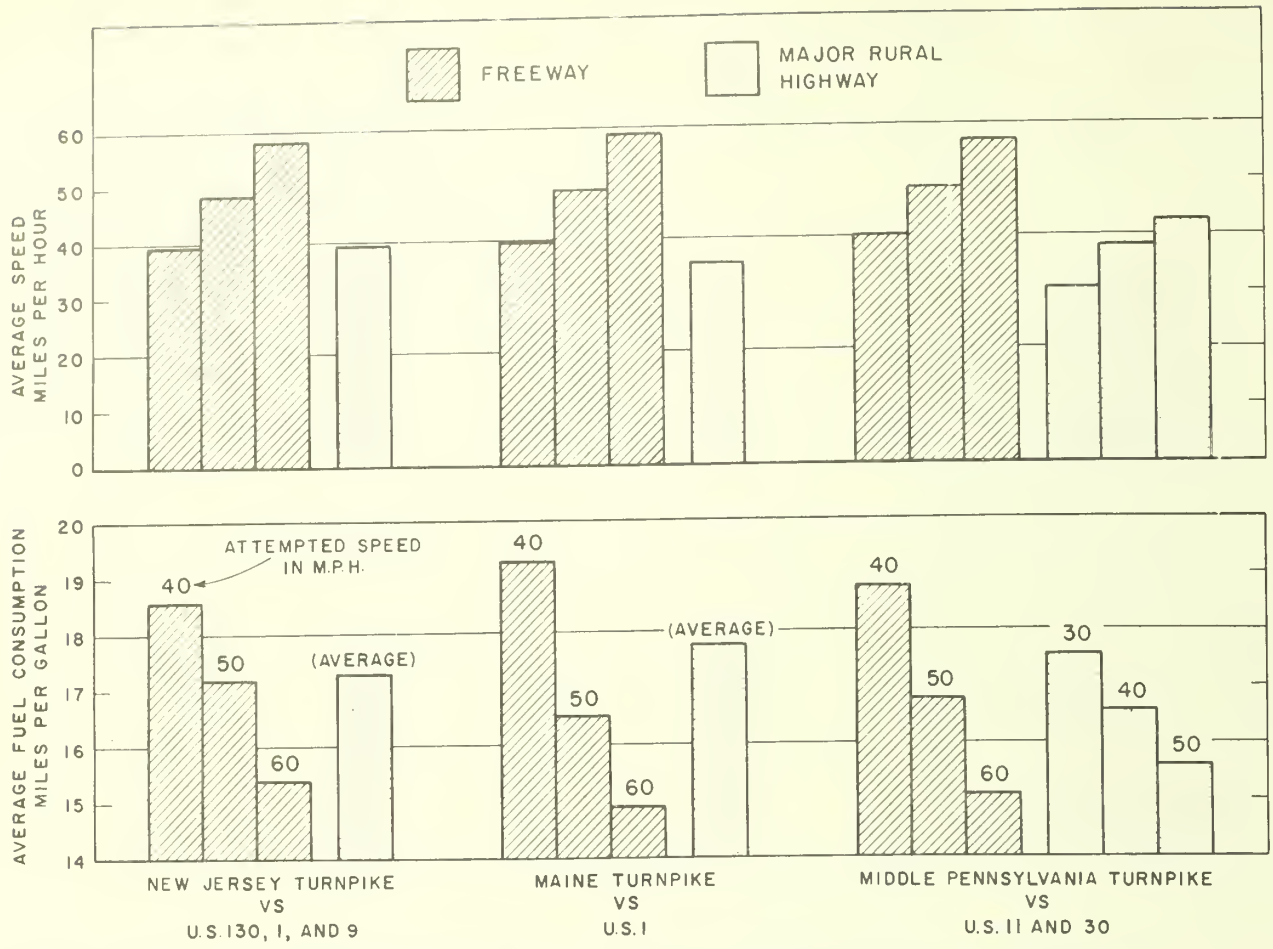

Figure 11.—Fuel consumption and speed of test car on freeways compared with that on parallel major rural highways.

obtained by frequent engine tune-ups, no attempt was made to correct the results for changing fuel-consumption characteristics. The triangular shaped points are the rates of consumption observed before the start of the project when there were 1,392 miles on the odometer and the engine was apparently either not properly "broken-in" or tuned.

In the fall of 1953, about one year after the completion of the freeway and special studies, it was planned to make some special grade tests with the same passenger car. The vehicle was calibrated at that time, and the rates of consumption, indicated by the circular points in figure 9, were found to be less than the minimum rates observed for the previous period of tests. For this reason, the engine was given a tune-up that included the replacement of spark plugs, and overhaul of carburetor and distributor. The rates of consumption observed after this tune-up, indicated in figure 9 by the square-shaped symbols, fell generally on or above the average curve and well within the band created by the maximum and minimum lines.

Calibration of instruments

The accuracy of the instruments for measuring deceleration, throttle opening, and intake manifold vacuum was checked only a few

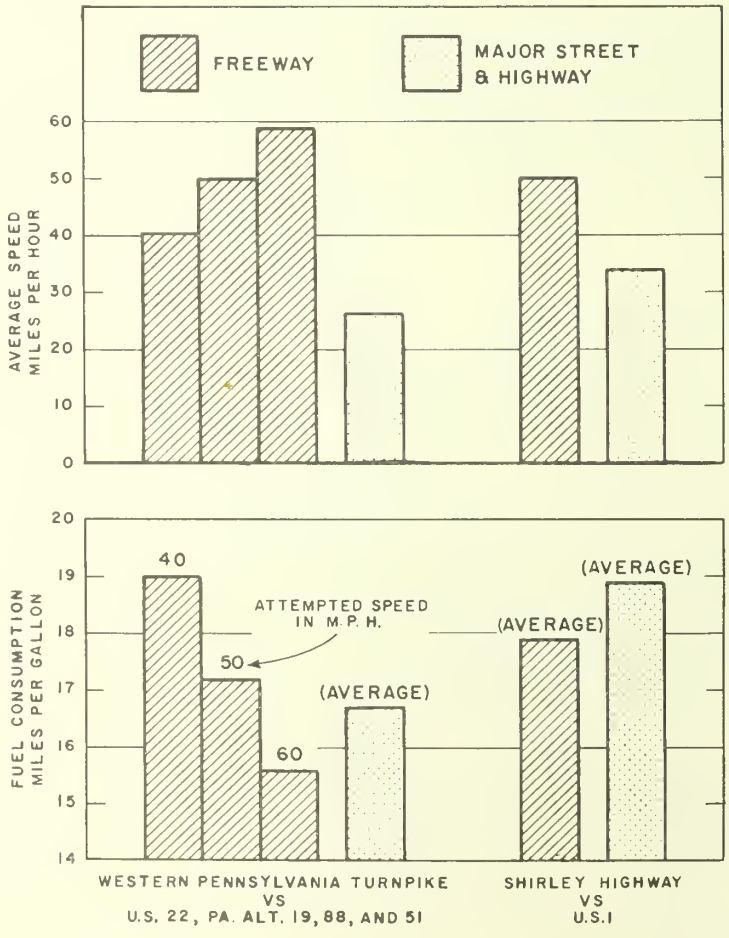

Figure 12 (Right).—*Fuel consumption and speed on freeways compared with that on parallel major streets and highways. A sizable percentage of the latter mileage is in urban areas.*

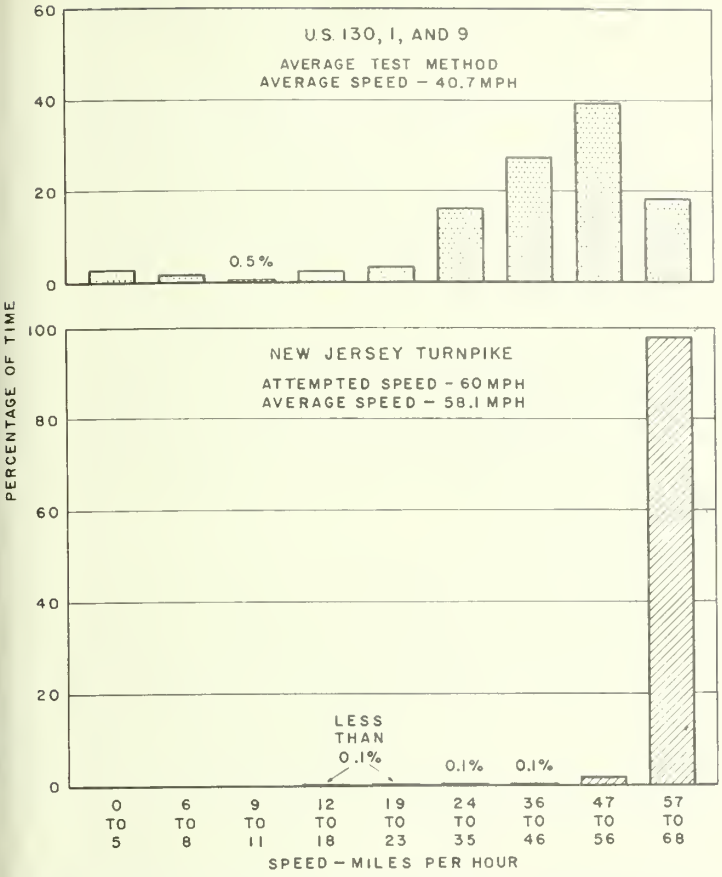

Figure 13.—Time distribution by speed groups for the New Jersey Turnpike and parallel major highway.

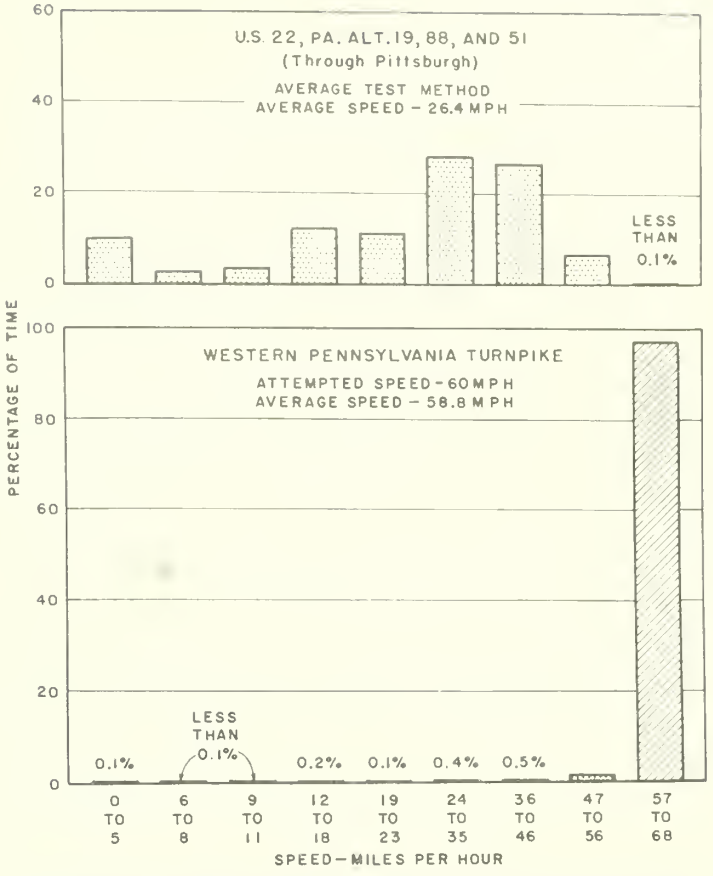

Figure 14.—Time distribution by speed groups for the western Pennsylvania Turnpike and parallel major highway.

times during the entire series of studies. However, the speedometer was calibrated frequently against the test car speedometer which in turn had been calibrated with an accurate speedometer actuated by a test wheel. It was found that the class intervals originally established for a given bank of counters did not vary appreciably during the tests.

The volumetric fuelmeter, which was of the positive displacement type, was calibrated in conjunction with the fuel calibration of the test vehicle before and after most of the studies. The results of the calibration tests, made with a burette that could be read to the nearest cubic centimeter, are shown in figure 10. These tests were conducted on a one-mile level section of highway at the indicated speeds. A plus error indicates that the fuelmeter reading in gallons was less than the true consumption. Of course, the opposite was true for a negative error.

Since speed is proportional to the rate of flow, it is evident in figure 10 that the fuelmeter did not give the same accuracy for all rates of flow. The fuelmeter was purposely adjusted to give the higher degree of accuracy for flow rates comparable to those for sustained speeds of 30 miles per hour or more, because rates of flow in that range were normally required. The average error was decidedly on the plus side for the lower flow rates and slightly on the negative side for the higher flow rates; it increased at a fast rate as the flow decreased below the flow rates comparable to speeds of 30 miles per hour or less. The fuelmeter reading will result in a rate of consumption that is considerably lower than the true rate, if the engine operates at or near idle speed for an appreciable portion of the total running time.

The results of the calibration tests were

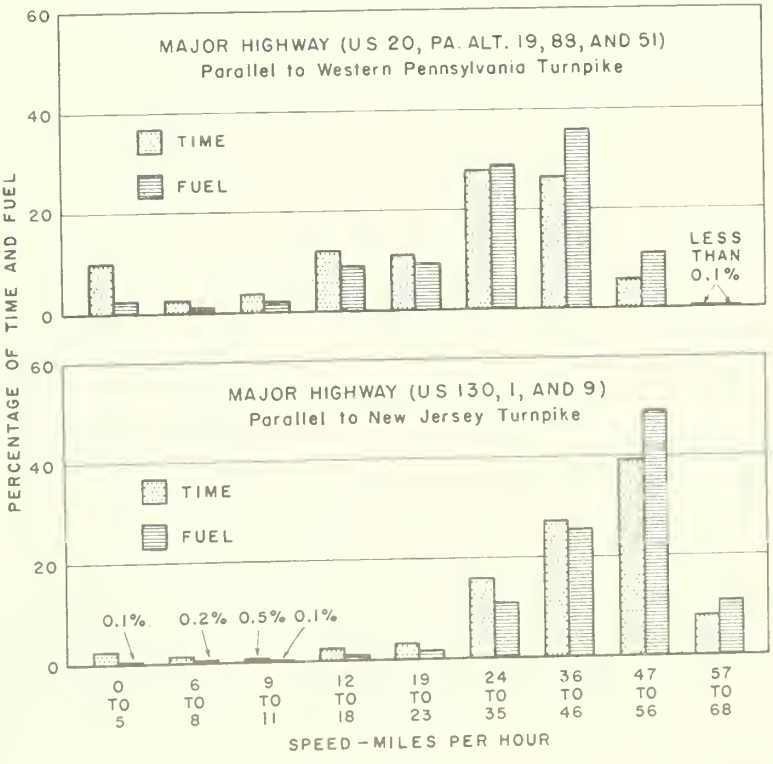

Figure 15.—Comparison between time and fuel distribution, by speed groups, for major highways parallel to the New Jersey Turnpike and western Pennsylvania Turnpike.

used to correct the observed rates of consumption to a common base, if it could be determined that the flow rates were consistently high. Correction factors could not be developed for those tests with considerable low-speed operation, since it was not possible from the speed record obtained on the counters to ascertain whether the vehicle was accelerating with a high flow rate or idling with a low flow rate. The variation in fuelmeter accuracy during a study was not of sufficient magnitude to affect materially the relative fuel consumption for two parallel routes studied at approximately the same time. However, it was necessary to correct to a common base in order to relate the results of the various studies, since the accuracy of the fuelmeter is shown in figure 10 to vary appreciably during the period of the studies.

Table 3.—Comparison between fuel consumption and travel time of test vehicle on freeway and on parallel major highway

Test route	Length of route		Average overall travel speed		Average rate of fuel consumption		Freeway-major highway ratio	
	Major highway	Freeway	Major highway [1]	Freeway [2]	Major highway	Freeway [3]	Travel time	Fuel consumption
	Miles	*Miles*	*M. p. h.*	*M. p. h.*	*M. p. g.*	*M. p. g.*		
New Jersey Turnpike	122.2	116.3	38.3	55	17.4	16.0	0.66	1.03
Pennsylvania Turnpike, middle section	[4] 163.0	[4] 159.7	42.7	57	15.6	15.1	.73	1.01
Maine Turnpike	43.8	41.8	35.7	55	17.8	15.7	.62	1.08
Pennsylvania Turnpike, western extension	58.5	55.2	26.4	57	16.7	16.0	.44	.99
Shirley Memorial Highway, Va	20.3	18.4	33.8	50	18.9	17.9	.61	.96

[1] Results of using average test method, except for middle section of Pennsylvania Turnpike where attempted speed test method of 50 m. p. h. was used.

[2] Based on available reports of average overall travel speeds of passenger cars, except for Shirley Memorial Highway where average test method was used.

[3] Interpolated from results determined by attempted speed test method, except for Shirley Memorial Highway where average test method was used.

[4] Distance between Middlesex and Irwin interchanges.

Special test procedures

In order to determine the relation between fuel consumption, speed, and degree of gradient, the test car was operated at sustained speeds ranging from 15 to 70 miles per hour on a 0.0-, 2.84-, 6.0- and 8.0-percent grade. For each sustained speed, at least three runs were made in both directions over a given grade. The fuel consumed by the test car was measured with a graduated burette which was connected in the fuel line between the car fuel pump and the carburetor. Fuel was pumped by the regular fuel pump into the burette and by an electric fuel pump from the burette to the carburetor. The temperature of the fuel in the burette was recorded for each run. Because the range of these temperatures was small, no attempt was made to correct the observed volumes to a standard base.

The accelerating ability of the test car was measured on the same four grades. Test runs were made with wide-open throttle in each direction on each test section, accelerating through each gear from a standing start to about 40 miles per hour, and in direct gear (third) from a speed of 20 miles per hour to the highest practicable speed. A minimum of two test runs was made for each condition of test.

The acceleration was determined from a record of time and distance, which was made on wax-coated paper fed through a chronograph at a constant speed of about 5 inches per second. Time was recorded on the tape at 1-second intervals by a small electrically actuated hammer wired to a timer. The record of distance was obtained by means of a rotating contact housed on a test wheel and driven by an odometer shaft. The rotating contact opened and closed an electrical circuit at every 2 feet of travel causing a stylus of the chronograph to make a crenelated trace on the moving tape.

A time-distance curve was plotted for each test run. This curve was differentiated by the mirror method at frequent points to determine instantaneous speeds. After the first differentiation, a time-speed curve was plotted and differentiated to obtain approximate instantaneous rates of acceleration. From these results, it was possible to derive relations for each grade that could be used to determine the distance and time required to accelerate between any two speeds, and the instantaneous acceleration rates for given speeds.

In conjunction with the acceleration tests, the fuel consumed while accelerating was measured with the burette at frequent points during each test run. When the burette was read, the chronograph tape was marked by pushing a switch wired to a stylus. It was then possible to determine the speed at the instant the burette was read. The result was an accumulative record of fuel consumption by speed which could be used to find the fuel consumed when accelerating between any two speeds.

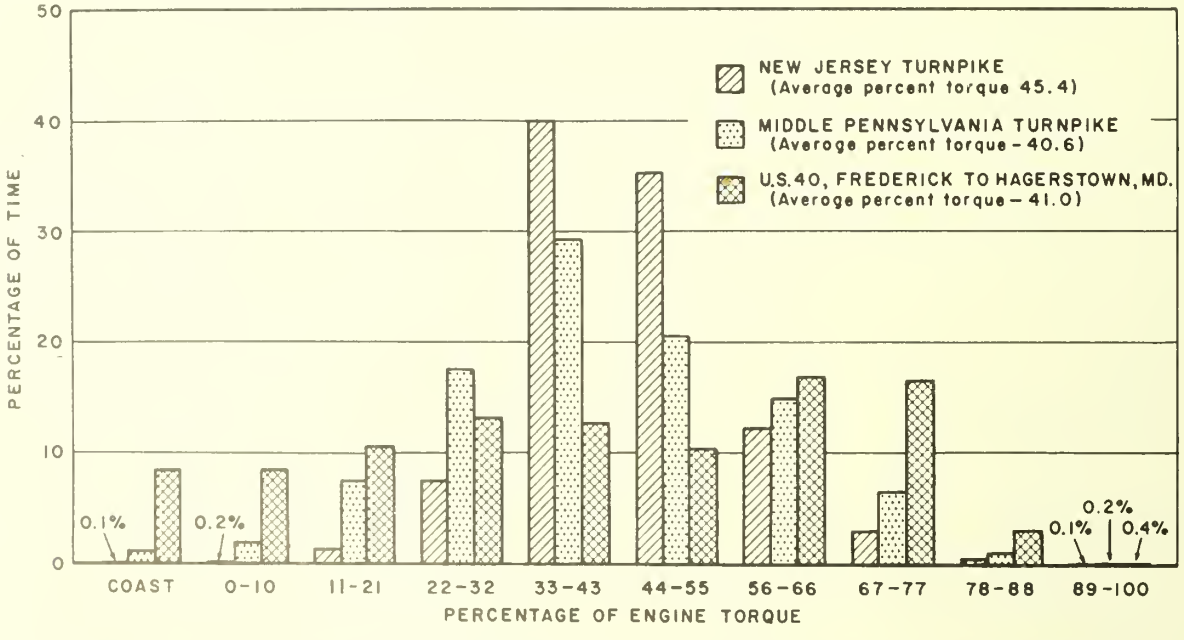

Figure 16.—Comparison of time distribution by percentage of engine torque for attempted speed of 60 m. p. h. on three test routes with different profile characteristics.

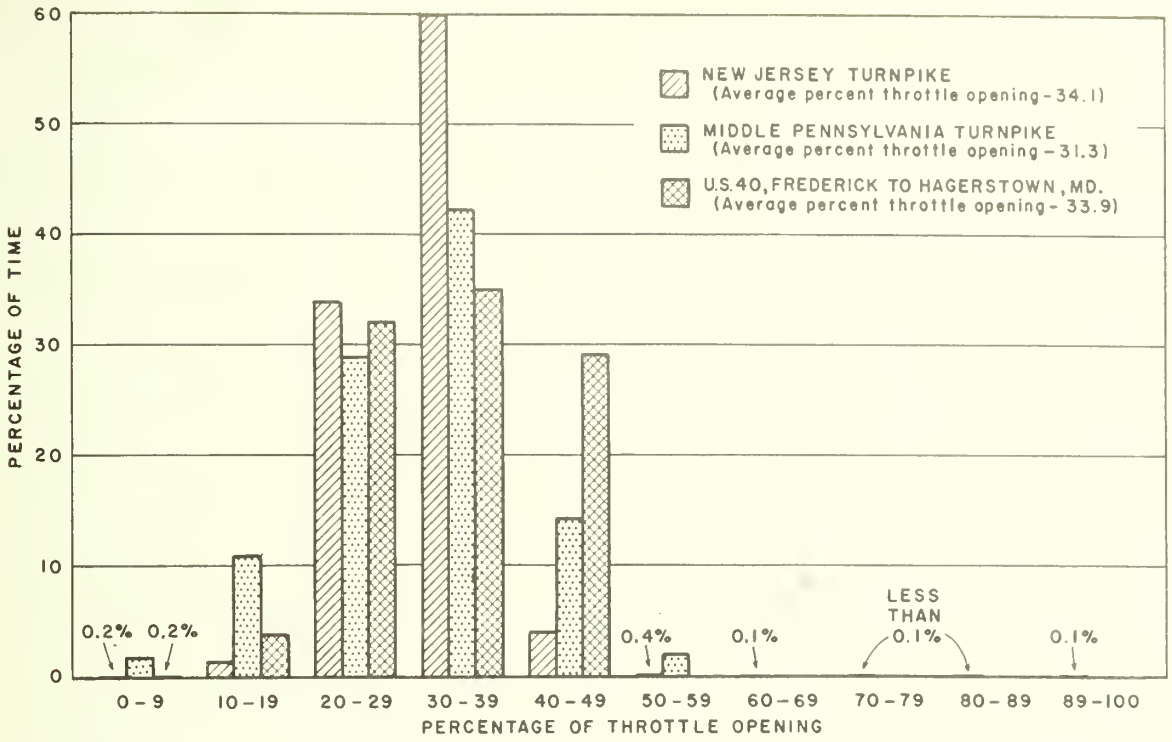

Figure 17.—Comparison of time distribution by percentage of throttle opening for attempted speed of 60 m. p. h. on three test routes with different profile characteristics.

The pertinent specifications of the test car are as follows:

Make and model	1951 Pontiac 6, 4-door sedan.
Transmission	3-speed synchromesh.
Weight:	
Front	1,920 pounds.
Rear	2,080 pounds.
Total	4,000 pounds.
Bore and stroke	3-9/16 × 4 inches.
Piston displacement	239.2 cu. in.
Compression ratio	6.5 to 1.
Transmission ratios:	
1st	2.67 to 1.
2d	1.66 to 1.
3d	1 to 1.
Rear axle ratio	4.10 to 1.
Maximum gross horsepower.	96 at 3400 r. p. m.
Maximum net horsepower.	90 at 3400 r. p. m.
Maximum gross torque	191 at 1200 r. p. m.
Maximum net torque	186 at 1000 r. p. m.

The horsepower and torque data, taken from curves in the Manufacturer's Shop Manual, are shown in table 1.

Summary of Basic Data

The results for each test route are summarized in table 2. This summary forms the basis for a discussion of the operating characteristics of the test car on freeways and the parallel major highways, and for a brief résumé of the findings for the four special studies. It contains the average rates of speed and fuel consumption, the average engine torque, and the average throttle opening for each test method (average or attempted speed). The average engine torque and throttle opening were determined by weighting the percentage of the total trip time recorded in each class interval with the midpoint value of the given class interval.

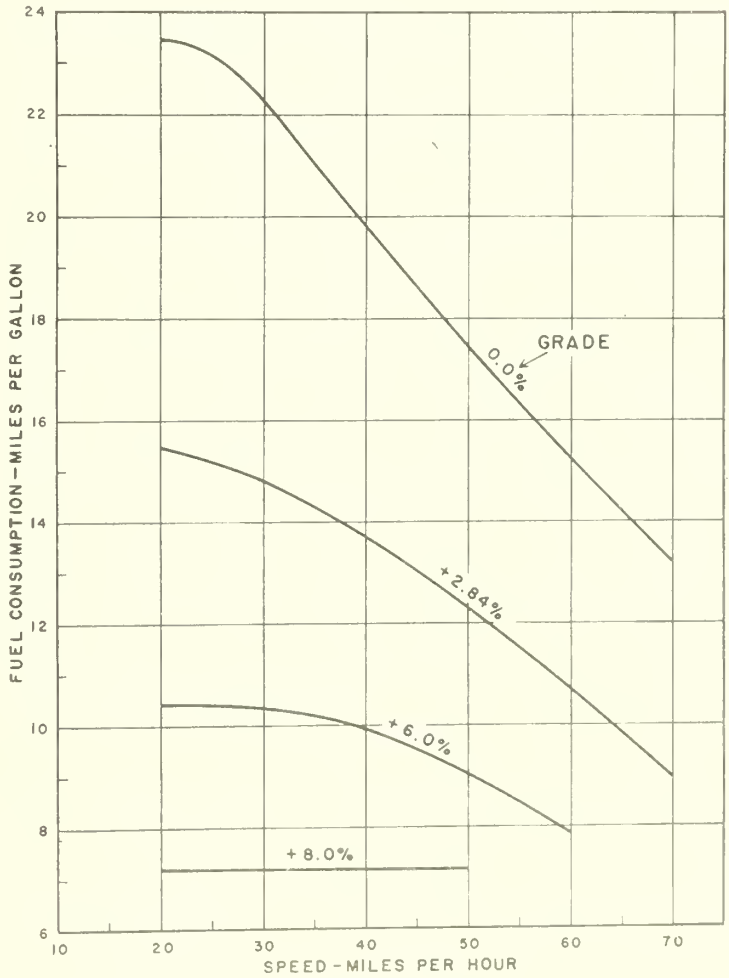

Figure 18.—Fuel consumption of test car ascending uniform grades at sustained speeds.

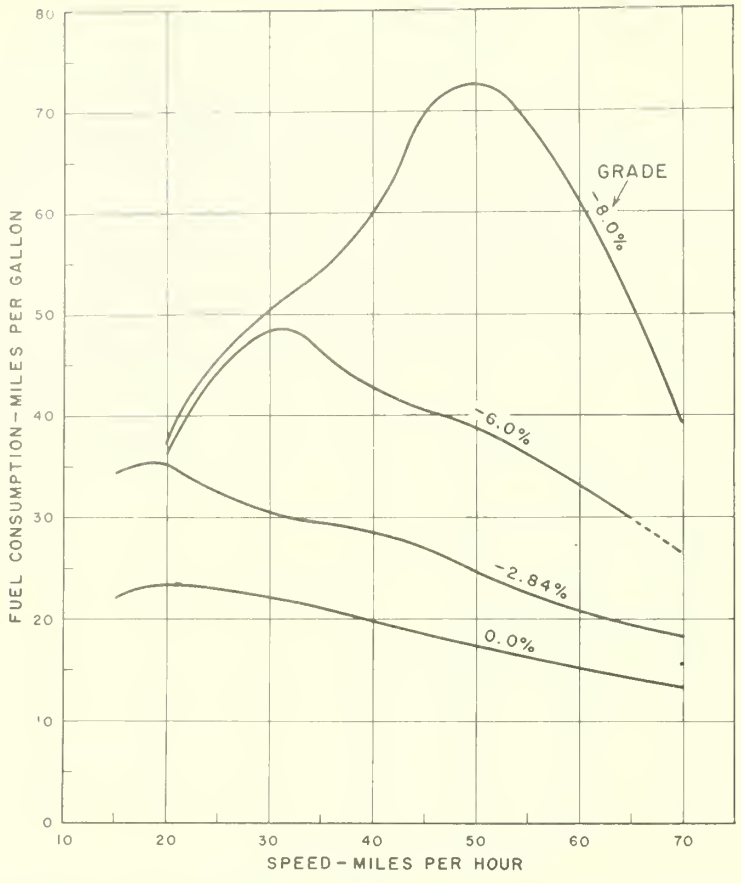

Figure 19.—Fuel consumption of test car descending uniform grades at sustained speeds.

Correction factors derived from the results of the fuelmeter calibration tests were applied to the observed rates of consumption to produce the values shown in table 2; except where no correction was warranted, and except in the cases of intersection and traffic light studies (facilities 8 and 9). In the latter instances, reliable factors could not be developed, because the test car operated a high percentage of the time at speeds less than 30 miles per hour.

Also included in table 2 are braking data which show the percentage of time spent in braking, the maximum class interval in which time was recorded, and a time factor. The vehicle was considered to be braking when the deceleration rate was more than 3 feet per second per second. The time factor is a ratio of the number of seconds recorded in class intervals of over 0–3 feet per second per second and the length of the test route in hundreds of miles.

The Freeway Studies

Speed and fuel consumption

The rates of fuel consumption and speed, shown in table 2 for freeways and parallel highways, are compared in figures 11 and 12. The term "average" over a bar indicates that the rate of fuel consumption or speed was obtained by driving the average test method. In figure 11, the three major highways are classed as rural, although they pass through numerous urban areas in New Jersey and Maine. The two parallel routes, identified in figure 12, include a substantial percentage of urban mileage.

For studies involving the New Jersey and Maine Turnpikes and the western section of the Pennsylvania Turnpike, the freeway was run with attempted speeds of 40, 50, and 60 miles per hour, and the parallel routes by the average test method. In the case of the middle Pennsylvania Turnpike study, both routes were run with the attempted speed test method—the freeway at speeds of 40, 50, and 60 miles per hour, and the major highway at speeds of 30, 40, and 50 miles per hour. The average test method was used for both the Shirley Memorial Highway and its parallel routes.

In this report it was assumed that the speed and fuel consumption rates observed on U. S. 11 and 30 in Pennsylvania for the attempted speed of 50 miles per hour approximated the performance that would have been obtained by the average test method. This was necessary because the traffic on many parts of this route was too light to use the average method of test. Values plotted in figure 11 for this route were based on the results which include the operations in the six major towns. The exclusion of these towns, as shown in table 2, increased the average speeds especially for the attempted speed of 50 miles per hour, but did not materially change the rates of fuel consumption.

From the comparisons in figures 11 and 12, except for the Shirley Memorial Highway, it is possible to gain an idea of the overall travel speeds that must be driven on the freeways to obtain a rate of fuel consumption that approximately equals that obtained by the average test method on the parallel route. In the case of the New Jersey and Maine Turnpikes the average speed is indicated to be less than 50 miles per hour, and for the middle and western sections of the Pennsyl-

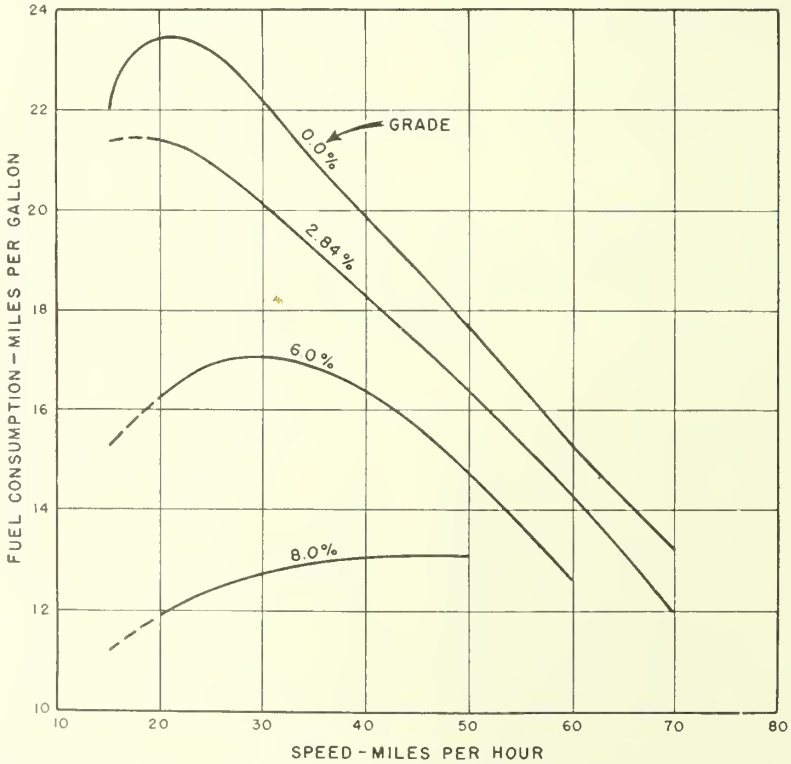

Figure 20.—Composite fuel consumption of test car ascending and descending uniform grades at sustained speeds.

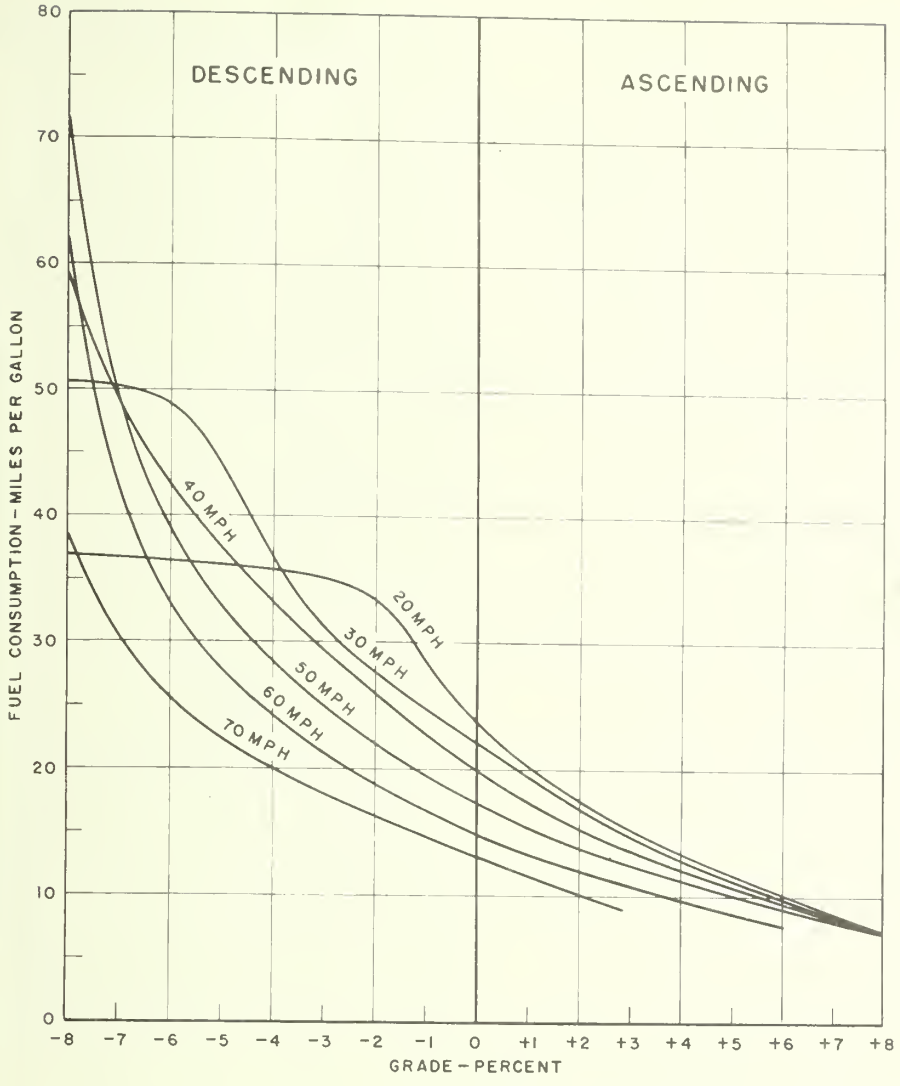

Figure 21 (Left).—*Directional fuel consumption of test car for various sustained speeds as related to gradient.*

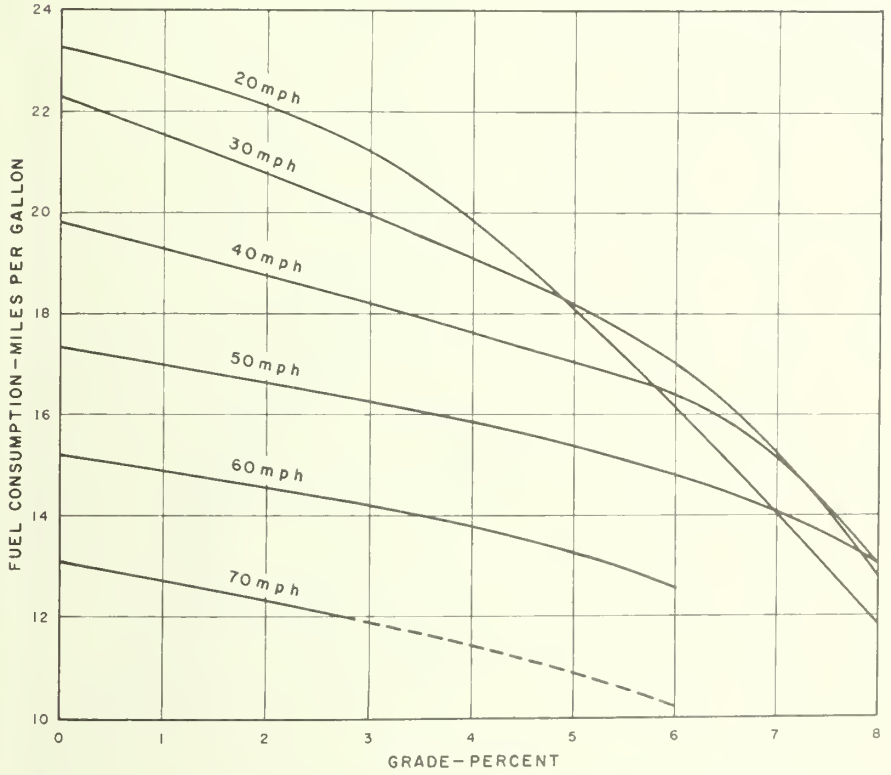

Figure 22.—Composite fuel consumption of test car, in miles per gallon, for various sustained speeds as related to gradient.

vania Turnpike it lies between 50 and 60 miles per hour. By actual interpolation of curves drawn to show the relation between the rates of fuel consumption and the average speeds obtained for the attempted speeds, the speeds which gave equivalent consumption rates were 48, 46, 54, and 53 miles per hour for the turnpikes in the order previously mentioned.

It is interesting to speculate on the reasons why the New Jersey and Maine Turnpikes must be traveled at slower speeds than the two sections of the Pennsylvania Turnpike in order to match the rates of consumption observed on the respective parallel routes. The principal reasons undoubtedly are that the major highway paralleling the middle Pennsylvania Turnpike has much more rise and fall than the routes which parallel the New Jersey and Maine Turnpikes, and the western Pennsylvania Turnpike involves considerably less traffic congestion with the resultant stop-and-go driving. The western section also has a small advantage over the parallel route in the degree of rise and fall.

By referring again to figures 11 and 12, it is seen that the average speed approximates the attempted speed in each instance. This fact indicates that very little traffic interference was encountered on the turnpikes up to an attempted speed of 60 miles per hour. Also, the rate of fuel consumption for a given attempted speed was nearly the same for each of the four turnpikes. For instance, the consumption rate for an attempted speed of 60 miles per hour was 15.4, 14.9, 15.1, and 15.6 miles per gallon for the New Jersey, Maine, and Pennsylvania Turnpikes, respectively.

Road-user benefits

The road-user benefits in terms of travel time and fuel consumption that might result from the use of the freeway by the test car are indicated in table 3. For this analysis the test car was assumed to operate at the average overall travel speeds of passenger cars on the four turnpikes—55 miles per hour for the New Jersey and Maine Turnpikes, and 57 miles per hour for the two sections of the Pennsylvania Turnpike. The rate of fuel consumption shown in table 3 for each of the four routes was based on these average speeds. In all other instances, the results used were obtained with the average test method, which was designed to produce an overall travel speed that approximated that of passenger cars using the facility.

The travel time ratios in table 3, which are based on the average overall travel speeds and the indicated lengths of the test routes, show that use of the freeway resulted in considerable time saving in each case. The ratios range from 0.44 for the western Pennsylvania Turnpike to 0.73 for the middle Pennsylvania Turnpike. In other words, the travel time on the freeway was 44 and 73

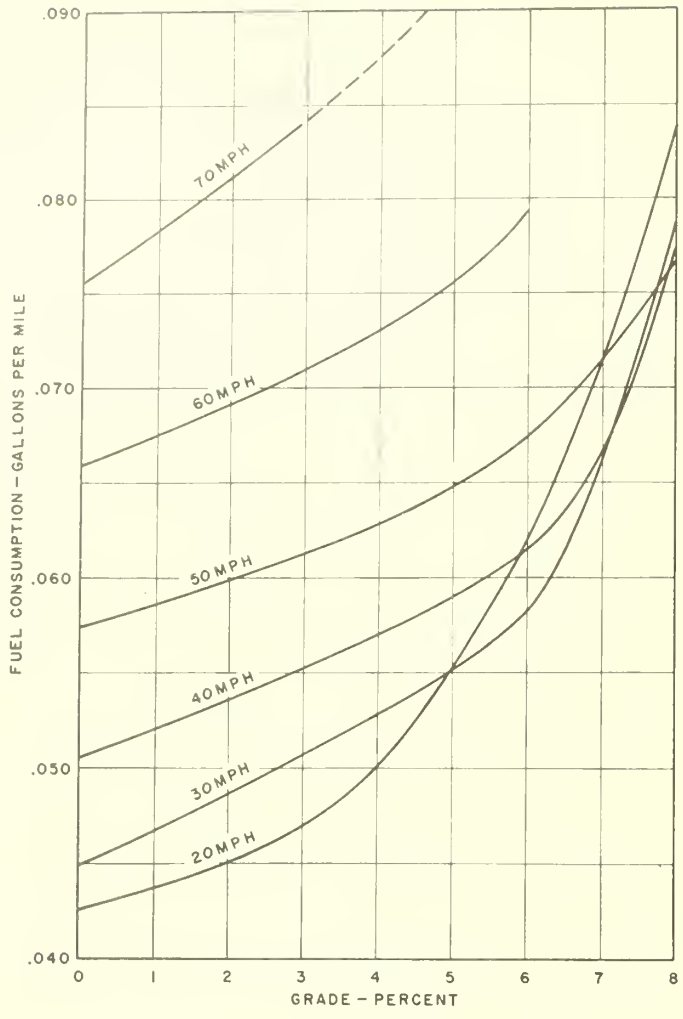

Figure 23 (Left).—***Composite fuel consumption of test car, in gallons per mile, for various sustained speeds as related to gradient.***

percent of that required on the respective parallel routes.

In contrast the fuel consumption ratios, computed from the average rates of consumption and distances reported in table 3, show that the test car would burn slightly more fuel on three of the freeways than on the parallel highways. This is indicated by a ratio greater than 1.00. The rates of consumption were higher on the freeway in each instance, although the difference was less than one mile per gallon for the two sections of the Pennsylvania Turnpike. Because of the distance saved by using the freeways, the consumption in gallons was about the same for each pair of routes with the possible exception of the Maine study, in which case the ratio was 1.08, an 8 percent advantage for the parallel major route.

The western Pennsylvania Turnpike study, reported in table 2, shows that the rate of consumption through the cities of Wilkinsburg and Pittsburgh (12.9 miles) averaged 14.9 miles per gallon, and through the 40.9-mile section, classed as urban, it averaged 16.5 miles per gallon. A comparison of these rates with that shown in table 3 for the parallel freeway definitely shows that considerable traffic congestion is required to increase the rate of consumption above that found at the normal overall travel speeds on the Pennsylvania Turnpike. Of course, a considerable

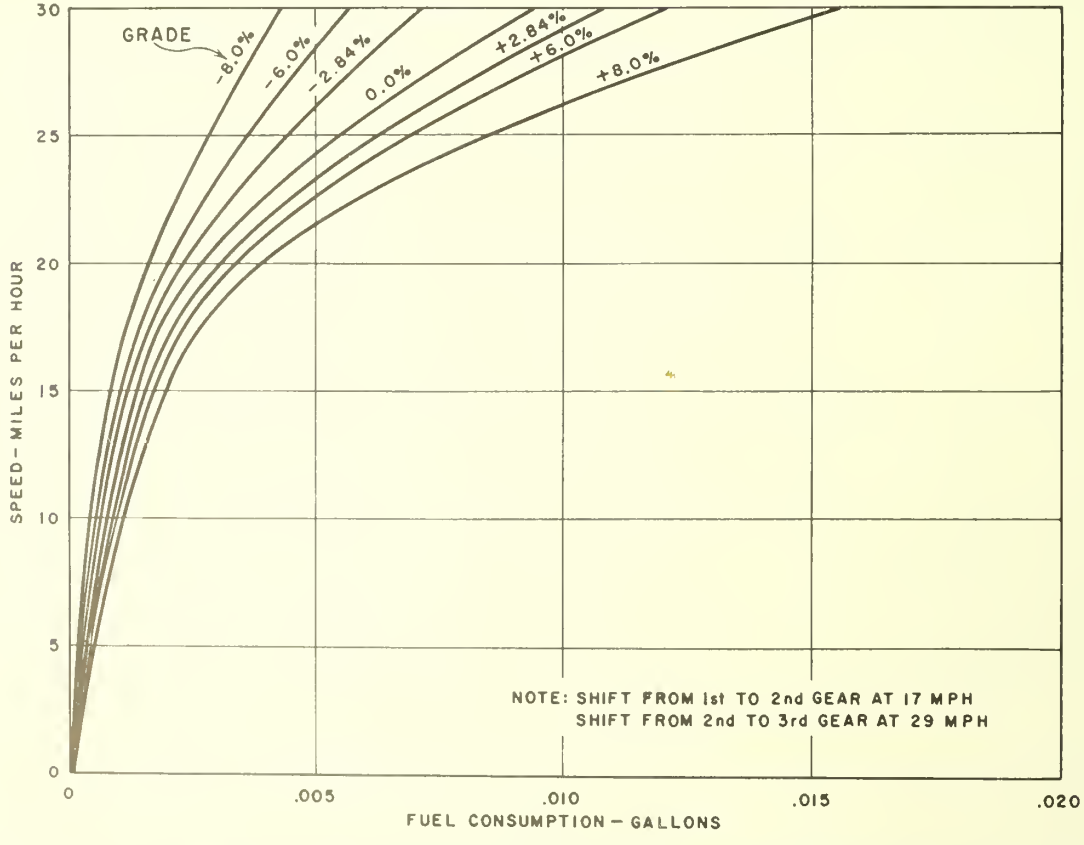

Figure 24.—Fuel required for test car to accelerate with full throttle through all transmission gears from a standing start to 30 m. p. h. on various upgrades and downgrades.

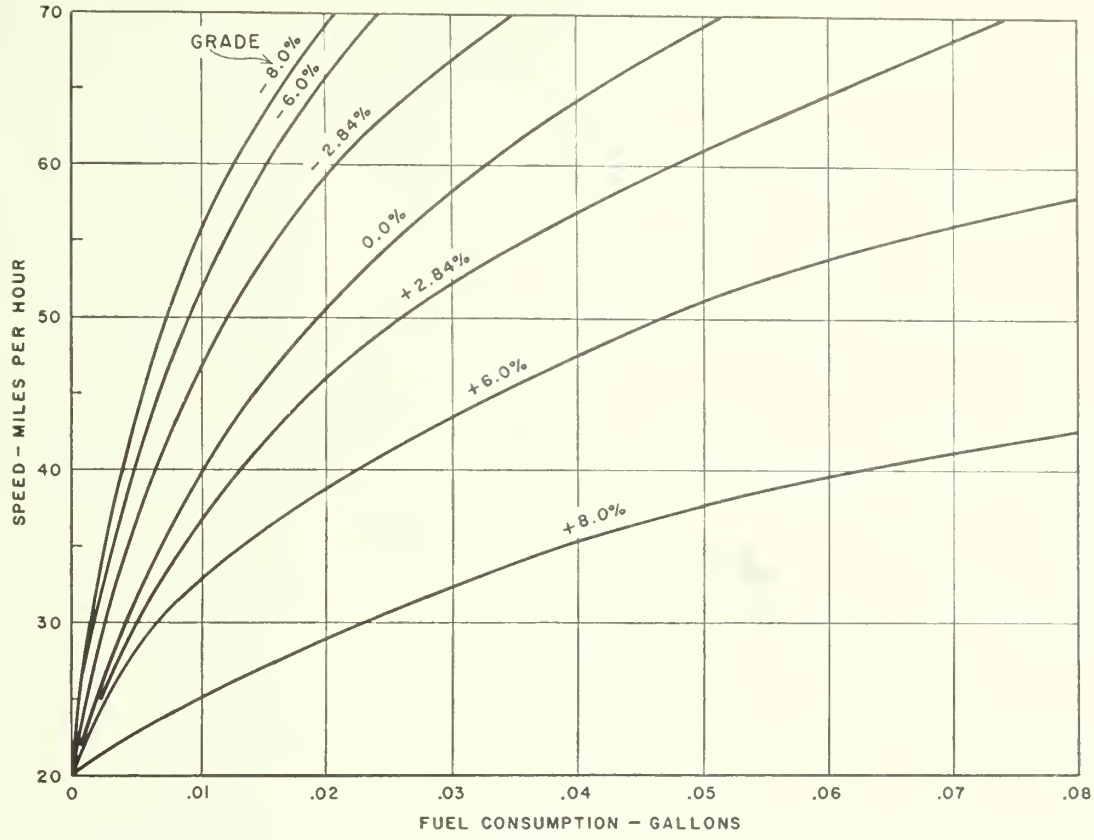

Figure 25.—Fuel required for test car to accelerate in third gear with full throttle from 20 m. p. h. to higher speeds on various upgrades and downgrades.

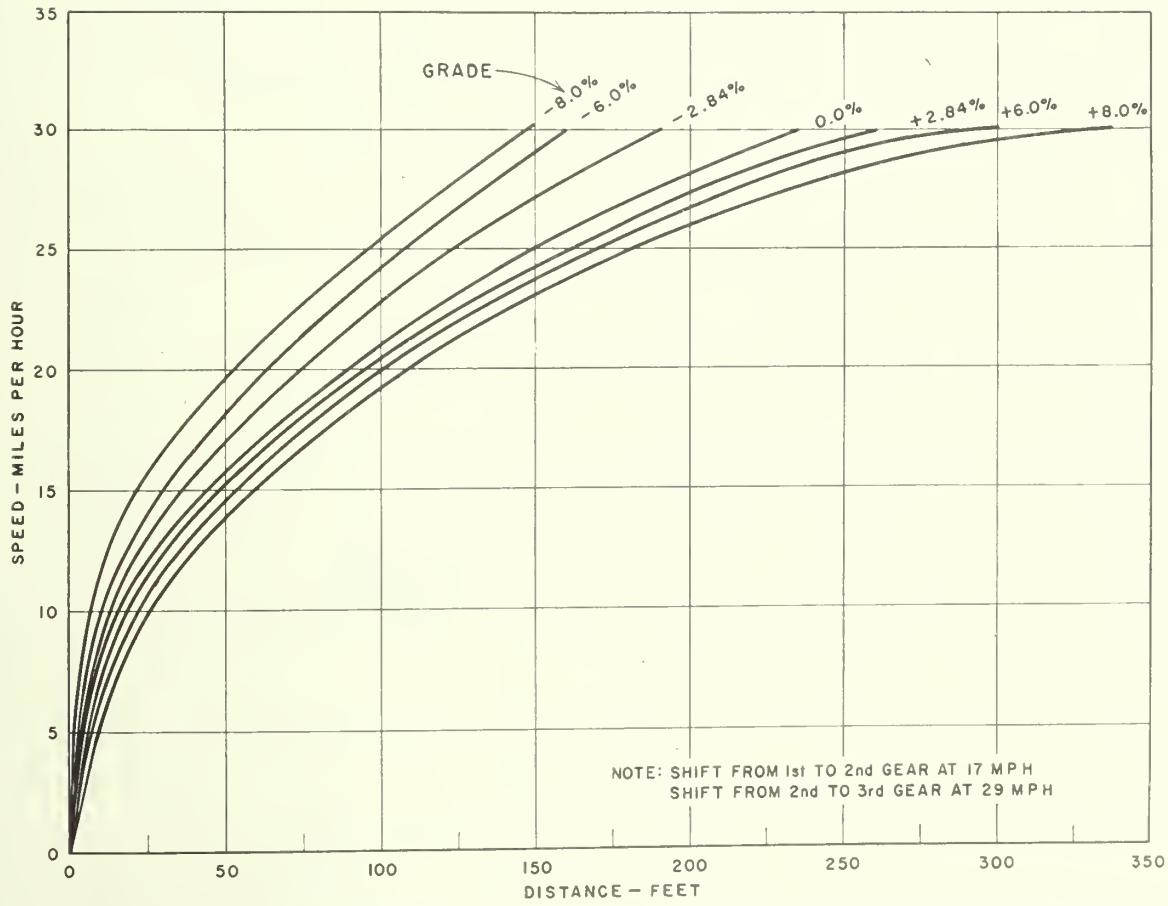

Figure 26.—Distance required for test car to accelerate with full throttle through all transmission gears from a standing start to 30 m. p. h. on various upgrades and downgrades.

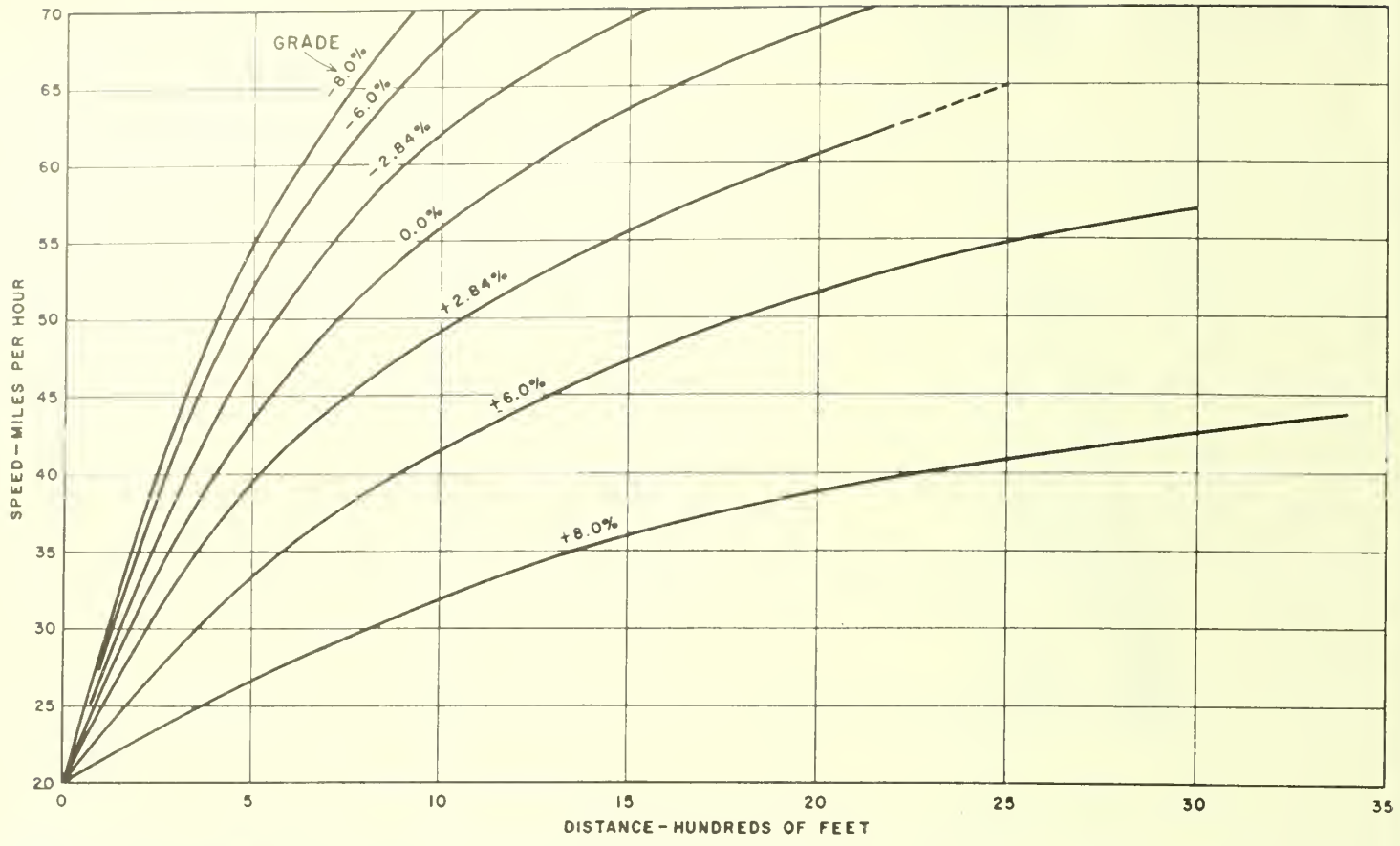

Figure 27.—Distance required for test car to accelerate in third gear with full throttle from 20 m. p. h. to higher speeds on various upgrades and downgrades.

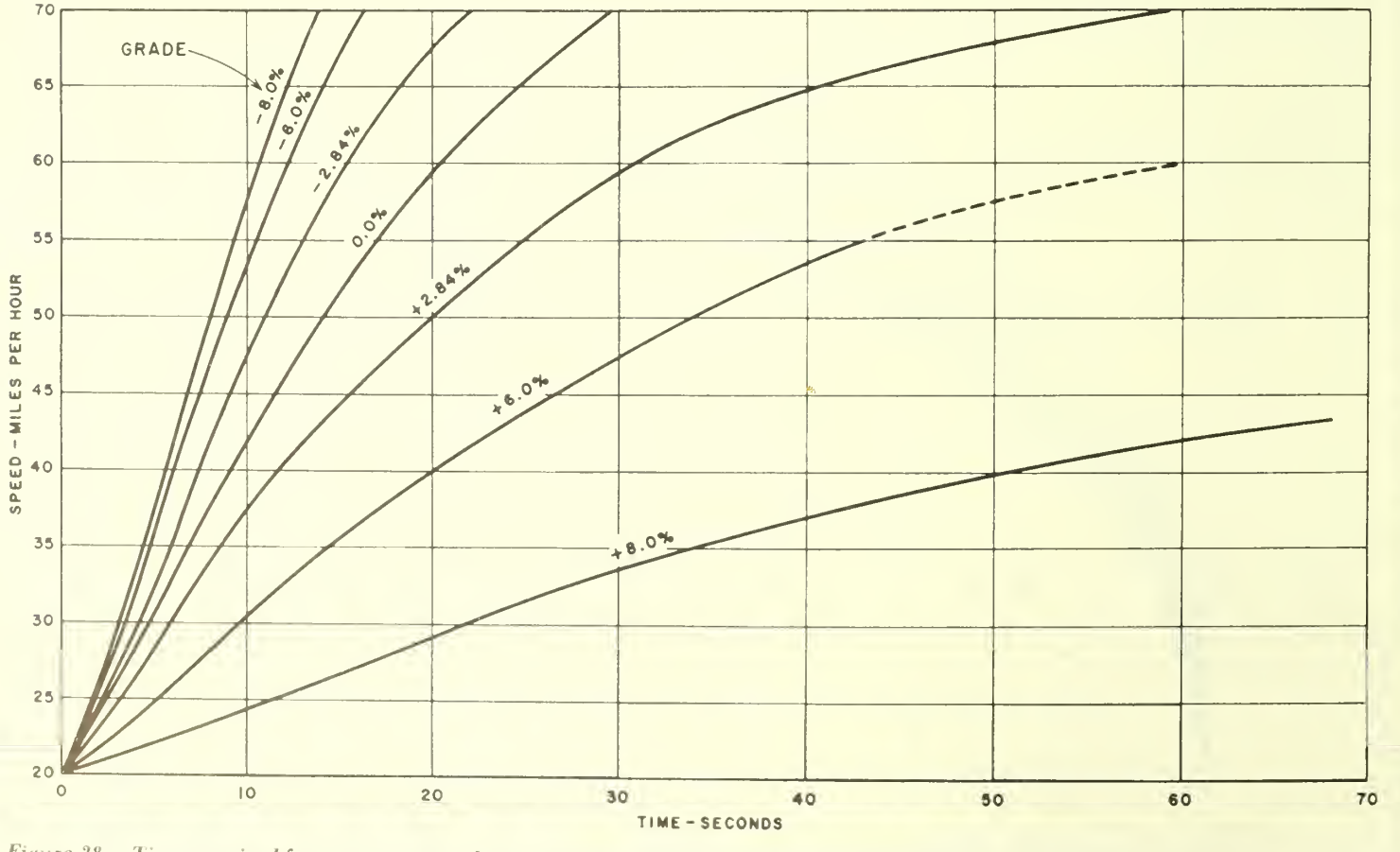

Figure 28.—Time required for test car to accelerate in third gear with full throttle from 20 m. p. h. to higher speeds on various upgrades and downgrades.

saving in fuel would be realized by operating at lower speeds on the Turnpike.

In the case of the New Jersey and western Pennsylvania studies, the parallel major highway was traveled before and after the opening of the turnpike. The results of these before and after studies are shown in table 2. They indicate that the opening of the turnpikes did not materially affect passenger car performance on the older routes.

Time and fuel distribution by speed

Two typical examples of the great contrast between vehicle operation on a freeway and on a major highway are shown in figure 13 for the New Jersey routes, and in figure 14 for the western Pennsylvania routes. In each of the two freeway examples, about 98 percent of the time for the attempted speed of 60 miles per hour was spent in the 57 to 68 mile-per-hour group. In the case of the parallel major highways the time was distributed over a much wider range which indicated a great number of speed changes.

There was also a considerable difference between the time distribution for the route paralleling the New Jersey Turnpike (figure 13) and for the route paralleling the western Pennsylvania Turnpike (fig. 14). In the former instance, about 9.6 percent of the time was spent at speeds below 24 miles per hour, and in the latter, the corresponding value was 38.9 percent. This wide variation in time distribution helps to explain the differences between the time and fuel consumption ratios shown in table 3 for the two sets of routes.

The distribution of time, shown in the upper portions of figures 13 and 14, is compared with the distribution of fuel in figure 15. An interesting point is the small percentage of fuel that was consumed below a speed of 24 miles per hour. On the route through Pittsburgh where the average speed was 26.4 miles per hour, only 23.9 percent of the fuel was burned below a speed of 24 miles per hour. About 10 percent of the time was spent in the 0 to 5 mile-per-hour class interval and only 2.5 percent of the fuel was used in the same class interval.

Built-in vehicle characteristics

One of the purposes of the study was to determine to what extent certain built-in vehicle characteristics were used in normal operation. The manner of conducting the tests precludes the use of speeds as a factor in this respect, except for the "average" runs made on parallel major highways. The percentage of time spent in each range of deceleration, engine torque, and throttle opening for the attempted speeds of 60 miles per hour, however, do indicate to some degree the normal use of brakes and power at average speeds slightly greater than the average overall travel speed of normal freeway traffic.

On the test routes which were operated with the average test method, the 57 to 68 miles-per-hour class interval was the highest speed in which any time was recorded. The percentage of time in this interval was less than 0.1 percent except for U. S. 130, 1, and 9 in New Jersey and the Shirley Memorial Highway in Virginia, where it was 8.0 and 7.4 percent, respectively.

The most surprising results are probably those shown in table 2 for the use of brakes. It is seen that the percentage of time spent in braking was practically nil for the freeways and rather insignificant for the parallel highways. The maximum deceleration recorded was in the range of 14–16 feet per second per second. Since the test vehicle by actual stopping-distance tests was capable of an average deceleration rate of 25.3 feet per second per second, only about 60 percent of the built-in braking force was used during any test.

Table 4.—Comparison of instantaneous acceleration rates for various speeds

Speed	Acceleration	
	Average vehicle	Test vehicle
M. p. h.	*M. p. h./sec.*	*M. p. h./sec.*
20	2.5	2.0
25	2.5	2.1
30	2.5	2.2
35	2.5	2.3
40	2.3	2.2
50	2.0	1.8
60	1.5	1.4
70	1.0	.8

Even though there was little time spent in braking on any route, a comparison of the time factors does indicate a sizable advantage for the freeways in this respect. For example, the time factor on the New Jersey Turnpike for an attempted speed of 60 miles per hour was 5.3 as compared with 181.2 for the parallel route before the opening of the Turnpike.

The average values of composite engine torque and throttle opening, shown in table 2, indicate that only a small portion of the built-in torque and power was normally utilized on any of the tests. This is emphasized by the time distributions shown in figures 16 and 17 for the three tests with the highest average engine torque and throttle opening. Time was seldom recorded in the highest two class intervals of engine torque (more than 77 percent) or in any class interval of throttle opening above 50 percent.

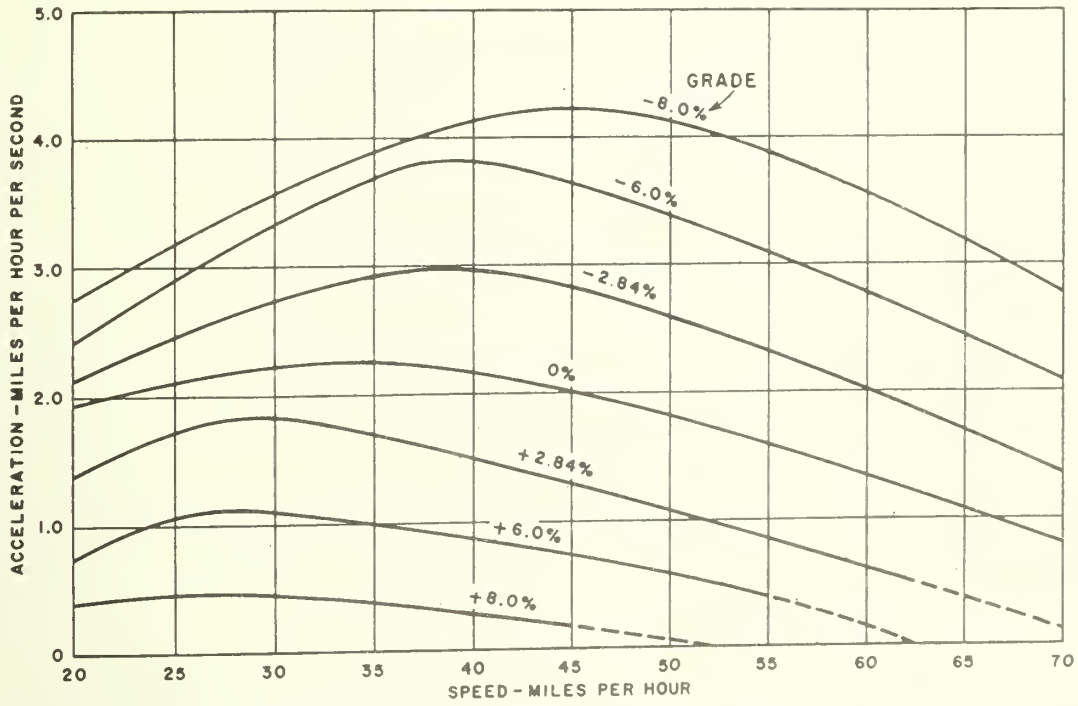

Figure 29.—Average instantaneous acceleration rates at various speeds for test car operating in third gear on various upgrades and downgrades.

The results shown in figures 16 and 17 were observed on three test routes with decidedly different profile characteristics. Operations on the New Jersey Turnpike were most consistent as indicated by about 75 percent of the time being spent in the engine torque range of 33 to 55 percent and about 90 percent of the time in the throttle opening range of 20 to 39 percent. In contrast, the time was distributed over a much wider range of both percentage of engine torque and throttle opening in the case of U. S. 40 which has a series of long steep grades.

Based on the data contained in table 2 and on the average overall travel speeds shown in table 3, the average engine torque and throttle opening observed on a major parallel highway were appreciably less than the average values observed on the corresponding freeway. For example, the average engine torque was 31.4 percent on U. S. 130, 1, and 9 in New Jersey and 41.2 percent by interpolation on the New Jersey Turnpike.

Résumé of Special Studies

U. S. 40 in Maryland

From a study made in 1947 between Hagerstown and Frederick, Md., it was found that the average speed of passenger cars was 33.6 miles per hour on the old section of U. S. 40 before the opening of the new section, and 42.5 miles per hour on the new section. For this reason the fuel consumption was measured on the old section by attempting to drive the average speed of 33.6 miles per hour in accordance with operating practices recorded at the time of the earlier tests. It is seen in table 2 that the average rate of fuel consumption was 16.6 miles per gallon on the old section at an average speed of 35.9 miles per hour. This rate compares with one of 17.1 miles per gallon determined for the average speed of 42.5 miles per hour by interpolating the rates measured on the new road for attempted speeds 40 and 50 miles per hour. The elimination of congestion created mostly by slow-moving trucks on steep grades appeared to result in a slight saving in fuel consumption.

Washington, D. C., to Annandale, Va.

The results shown in table 2 for this route are included in the report only for reference use, since the original purpose of the study has already been served.[3] The route which led to Annandale by way of the Shirley Memorial Highway was far superior in average speed especially during the peak traffic period. Also, the rate of fuel consumption by way of the Shirley Memorial Highway was lower during the peak period, 16.4 miles per gallon as compared with 15.4 miles per gallon, but approximately the same during the off-peak period.

The performance was not greatly reduced by heavier traffic on the freeway section, whereas it was materially reduced in the case of the section with intersections at grade. Also, the difference in performance on the two sections during the off-peak period was not great. It appears that sizable savings in fuel consumption may result in peak traffic periods through use of freeways under urban conditions of operation. This is, of course, contrary to the findings already reported for high-speed operations on freeways.

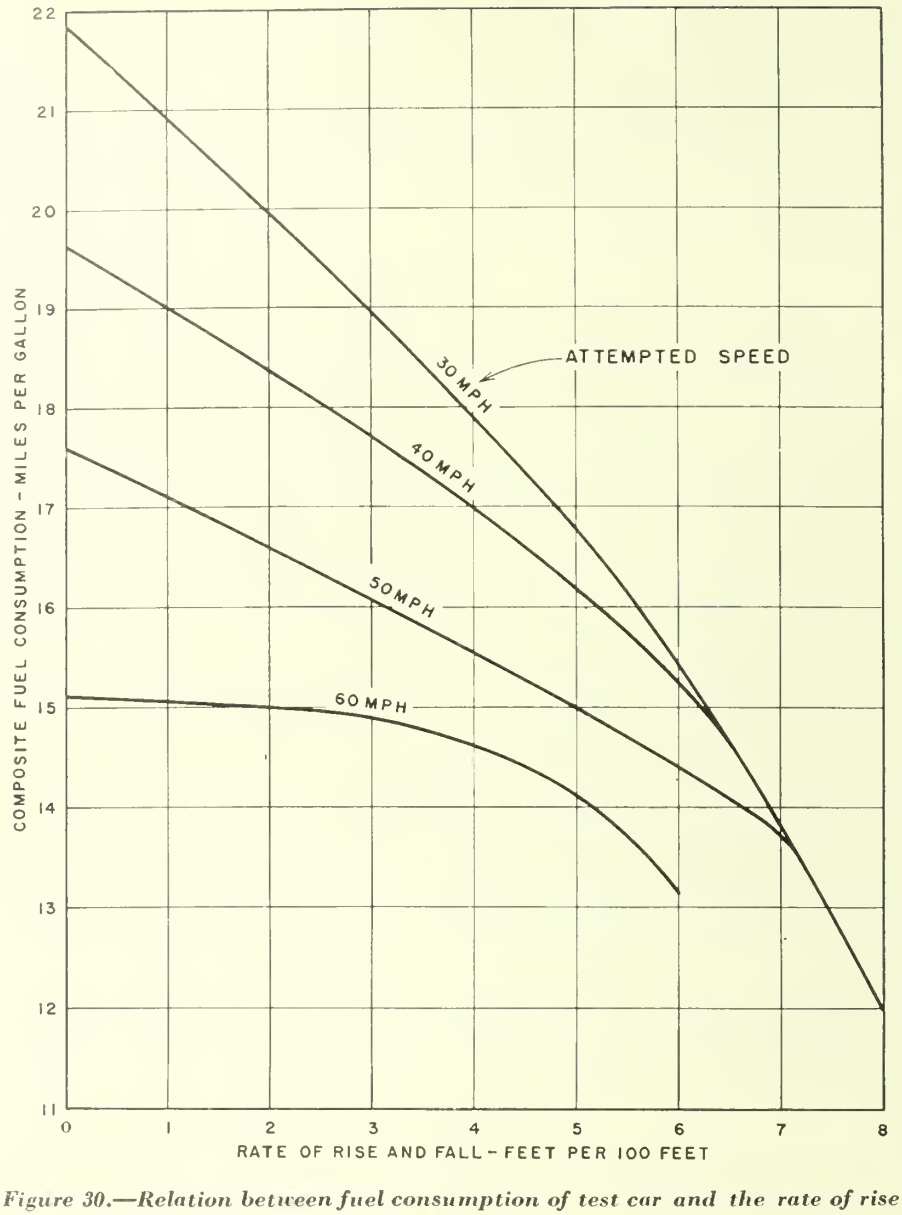

Figure 30.—Relation between fuel consumption of test car and the rate of rise and fall.

Intersection study

The results shown in table 2 (facility 8) need no explanation, except that the true rate of fuel consumption was probably somewhat higher than the value given in the table, because of the characteristics of the fuelmeter shown in figure 10. It was previously pointed out that the observed rates of consumption were shown in table 2 because reliable correction factors could not be derived for this predominantly low-speed operation.

Traffic light study

Tests were made before and after the installation of 11 traffic-actuated signals on the most congested section of Columbia Pike. The results are summarized in table 2 (facility 9). The comments just made about the rates of fuel consumption for the intersection study apply also to this study.

The pertinent findings were that the average overall travel speed was reduced about 5 percent and the rate of consumption was increased about 12 percent during the off-peak periods. During the peak period, the average overall travel speed was about the same but the rate of consumption was lower by about 6 percent. The purpose of the signal installation was to facilitate the cross traffic with as little interference as possible to the main traffic flow. If the movements of the cross traffic were expedited, as it would be reasonable to assume, it appeared that the purpose of the installation had been accomplished within reasonable limits.

Grade Test Results

Fuel consumption rates

In order to add to the scant data that have been reported for fuel characteristics of modern passenger cars on a wide variety of

[3] See footnote 1, page 182.

Table 5.—Savings in fuel consumption resulting from two methods of grade reduction

Grade reduction, in percent	Percentage of savings for sustained speeds of—							
	30 m. p. h.		40 m. p. h.		50 m. p. h.		60 m. p. h.	
	Method I [1]	Method II [2]	Method I	Method II	Method I	Method II	Method I	Method II
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
8–6	16.7	25.7	12.7	20.2	7.5	11.3		
8–4	15.2	33.4	10.9	26.0	6.0	17.6		
8–3	13.1	36.5	8.7	28.0	4.9	19.6		
8–2	9.8	39.0	6.6	30.4	3.7	21.6		
6–4	3.0	10.5	1.7	7.4	2.0	7.0	3.1	8.7
6–3	3.3	14.5	1.4	9.9	1.8	9.2	3.7	11.9
6–2	2.7	17.9	1.5	12.8	1.5	10.4	2.6	13.7
4–3	.9	4.5	.1	2.7	.3	2.4	1.3	3.5
4–2	1.1	8.3	.6	5.9	.5	4.8	.9	5.5
3–2	.4	3.9	.5	3.3	.3	2.4	0	2.0

[1] No reduction in rise and fall.
[2] Reduction in rise and fall.

gradients, the test car was driven on grades ranging from 0 to 8 percent. The vehicle was operated in direct gear at sustained speeds ranging from 15 to 70 miles per hour and was accelerated in various gears from a standing start to the highest practicable speed.

The rates of consumption in miles per gallon for the sustained speeds are shown in figure 18 for ascending, and figure 19 for descending four uniform grades. The composite consumption, which combines the results shown in figures 18 and 19, is given in figure 20. For the uphill tests, the fuel consumption decidedly increased at a slower rate with speed as the grade increased. This occurs mainly because air resistance, which increases approximately with the square of the speed, is constant for each grade and becomes a smaller portion of the total resistance to motion as the grade increases. It is seen that consumption remained almost constant for ascending the 8 percent grade, and actually decreased slightly with speed for the composite relation. The test car could not sustain a speed of 65 miles per hour on a 6-percent grade or 55 miles per hour on the 8-percent grade.

The directional fuel consumption shown in figures 18 and 19 and the composite fuel consumption shown in figure 20 are replotted in more usable form in figures 21 and 22, respectively. From these curves it is possible to determine easily the fuel consumption for any degree of gradient at a given sustained speed. In considering the composite consumption, the interesting point is that the rate of consumption increases at a fairly uniform rate with an increase in grade up to a grade of 6 percent for all except the 20-mile-per-hour sustained speed. Above 6 percent, the increase is at a faster rate which indicates that the reduction of grades above 6 percent should result in a saving in fuel consumption for the test vehicle, even if the rise and fall is not reduced. The relations for composite consumption shown in figure 22 are plotted in terms of gallons per mile in figure 23 for later use in this report.

Accumulative fuel curves for accelerating on the level and on various plus and minus grades with full throttle from a standing start to 30 miles per hour are shown in figure 24. Two gear shifts were made, one at 17 miles per hour and one at 29 miles per hour. Actually the vehicle operated in third (direct) gear only from 29 to 30 miles per hour. Similar relations for accelerating in third gear from 20 miles per hour to the highest practical speed are shown in figure 25. Since the fuel consumption is accumulated with speed, it is possible to determine from these data the fuel consumed for accelerating between any two given speeds.

These data should have application to the problem of estimating the cost savings that might accrue to users of passenger cars by the elimination of traffic congestion or other interruptions to the smooth flow of traffic, which cause the driver to accelerate from a reduced speed to the desired running speed. An example would be the economic analysis of the congestion caused by slow-moving trucks on hills.

Another useful value of fuel consumption obtained for the test car was the fuel consumed while idling. The consumption at an idling engine speed of approximately 460 revolutions per minute was 0.4 gallon per hour. At an engine speed of 600 revolutions per minute it was about 0.5 gallon per hour.

Acceleration rates

The distance required to accelerate with full throttle between any two speeds can be determined from the curves shown in figure 26 for accelerating through first and second gears from a standing start to 30 miles per hour, and in figure 27 for accelerating in third gear from 20 miles per hour to the highest practicable speed. For example, to obtain the distance required to accelerate up a 6-percent grade from 30 to 50 miles per hour, the accumulative distance of 350 feet at 30 miles per hour is subtracted from the accumulative distance of 1,800 feet at 50 miles per hour. The answer is 1,450 feet.

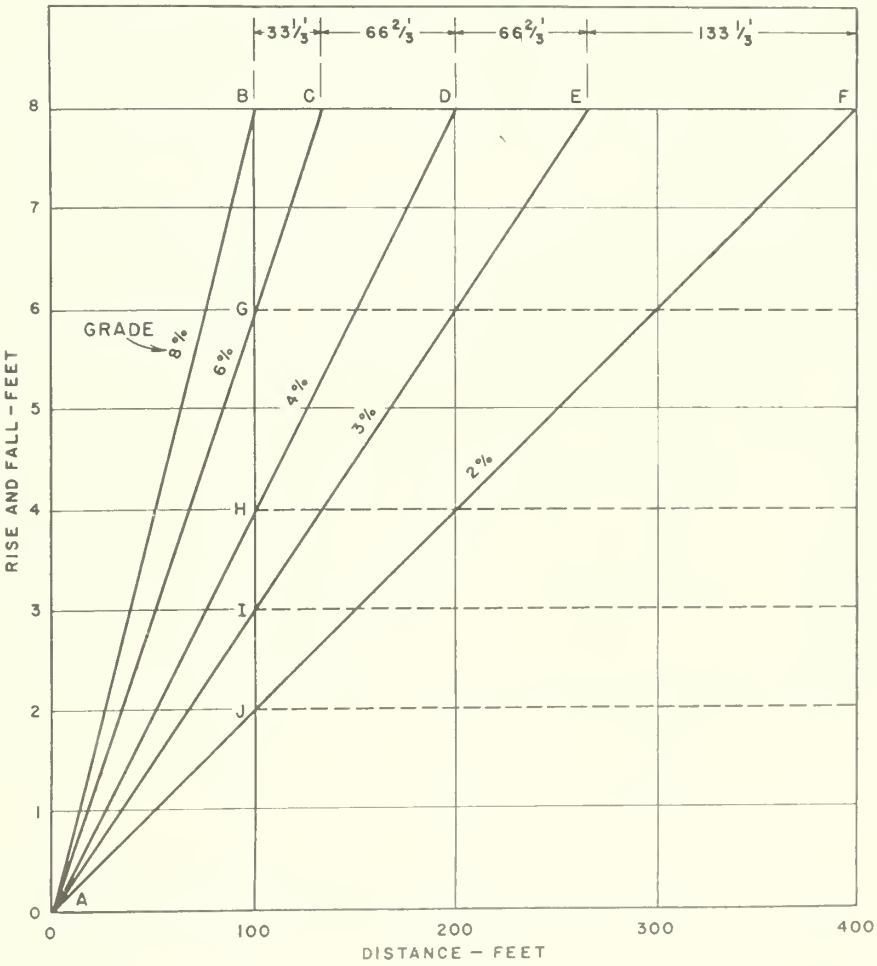

Figure 31.—Example for determining savings in fuel consumption by two typical methods of grade reduction.

Table 6.—Summary of fuel consumption between Frederick and Hagerstown, Md., measured and computed by various methods for attempted sustained speed of 50 miles per hour

Sections	Section length	Rise and fall rate	Fuelmeter measurement				Individual grade method	Rise and fall method		Grade classification method
			July 1951	Aug. 1952	Sept. 1952	Average		Rise and fall relation	Individual grade relation	
	Miles	*Ft./100 ft.*	*Gal.*	*Gal.*	*Gal.*	*Gal.*	*Gal.*	*Gal.*	*Gal.*	*Gal.*
A–B	3.5	3.8	0.200	----	0.220	0.210	0.224	0.223	0.219	0.223
B–C	1.8	4.5	.106	----	.117	.112	.119	.118	.115	.118
C–D	4.1	3.8	.231	----	.252	.242	.264	.262	.256	.266
D–E	2.4	5.7	.149	----	.160	.154	.167	.164	.160	.165
E–F	2.6	5.2	.156	----	.167	.161	.173	.174	.170	.172
F–G	6.6	2.2	.368	----	.390	.379	.399	.400	.398	.399
Total	21.0	3.7	1.210	1.318	1.306	[1] 1.278	1.346	[2] 1.333	[2] 1.310	1.343
Variation from burette measurement [3] pct.			−5.5	+3.0	+2.0	−0.2	+5.2	+4.9	+2.3	+5.0

[1] Not a summation of values for intermediate sections.

[2] Based on rate of rise and fall for total section, and not a summation of values for intermediate sections.

[3] Burette measurement, August 1952, 1.280 gallons.

Similar relations between speed and accumulative time are shown in figure 28 for the same plus and minus grades. The time required to cover the distance of 1,450 feet was determined to be approximately 24 seconds.

The relations in figures 25 and 27 may be used to determine the average rate of fuel consumption for accelerating between two speeds. For full throttle acceleration on a plus 6-percent grade from 30 to 50 miles, the rate was 6.9 miles per gallon. This was determined by dividing the distance in miles (fig. 27) by the fuel in gallons (fig. 25). The rate of 6.9 miles per gallon compares with one of 9.0 miles per gallon, read from figure 18 for a sustained speed of 50 miles per hour on an upgrade of 6 percent.

The instantaneous acceleration rates at various speeds are shown in figure 29. The peak acceleration on the level occurs at a road speed of 35 miles per hour which approximates the speed of peak torque. The shape of the acceleration curve is similar to the shape of the maximum torque curve and this should be the case, since acceleration is proportional to torque. The acceleration rates for the test vehicle are very similar to those obtained in a previous study for an average of 53 vehicles.[4] A comparison of the instantaneous rates for various speeds of the test vehicle and for the average of 53 vehicles is shown in table 4.

Analyses of Fuel Consumption

Rise and fall relations

The relations between fuel consumption and rise and fall, shown in figure 30 for attempted speeds of 30, 40, 50, and 60 miles per hour, were derived from the rates of composite fuel consumption observed on the individual test sections of the New Jersey Turnpike, Maine Turnpike, Pennsylvania Turnpike (both sections), Shirley Memorial Highway, U. S. 30 and 11 in Pennsylvania, and U. S. 40 in Maryland. If the average speed for a test section was not within about 5 percent of the attempted speed, the rate of fuel consumption was not used in this analysis.

The average curves shown in figure 30 for 30, 40, 50, and 60 miles per hour were based on 35, 79, 74, and 46 observations, respectively. There was a rather wide dispersion of the observed points about each of the curves. The standard errors of estimate in miles per gallon were 0.76 for 30 miles per hour, 0.79 for 40 miles per hour, 0.63 for 50 miles per hour, and 0.35 for 60 miles per hour. Part of the wide scatter of data about the curves was undoubtedly due to variations in performance of the test car during the period of tests, shown previously in figure 9. Another factor contributing to the large deviation was the inability to develop reliable correction factors for the varying accuracy of the fuelmeter, shown in figure 10.

The relations established between the rate of rise and fall and the rate of fuel consumption were very similar to those shown in figure 22, which were determined for sustained speed operation on short uniform grades. They provide a rather easy method for estimating the fuel used on any section of road. The particular advantage is that any combination of grades can be considered at one time by determining the total rise and fall for the highway section. A disadvantage is the error that results when the length of the very steep grades is an appreciable portion of the total length being considered. This error results because the composite effect of one foot of rise and fall, as shown in figure 30, is appreciably greater for the rates of rise and fall above 6 feet per hundred feet. The rate of fuel consumption was also shown in figure 22 to increase at a faster rate for grades over 6 percent.

Grade reduction methods

The savings in fuel consumption that result by reducing grades without a reduction in rise and fall, and with a reduction in rise and fall are indicated in table 5. They were computed from the example shown in figure 31 and the rates of fuel consumption (gallons per mile) shown in figure 23. In order to clarify the mechanics of the analysis, the problem of reducing an 8- to a 4-percent grade is described in detail for a speed of 30 miles per hour.

Figure 31 shows that if the reduction of the 8-percent grade is accomplished without a reduction in rise and fall, the saving in fuel would be the sum of the consumption on the 8-percent grade (AB) and the level section (BD), minus the consumption on the 4-percent grade (AD). The fuel consumed was 0.001983 gallon on AD (200 feet), 0.001491 gallon on AB (100 feet), and 0.000849 gallon on BD (100 feet). These values of consumption were determined by multiplying the length of the respective section in miles by the rate of consumption read for the specified grade from the 30-mile-per-hour curves in figure 23. The saving in fuel is thus 0.000357 gallon. The percentage of savings is 0.000357 gallon divided by 0.002340 gallon, or 15.2 percent.

If the reduction in the 8-percent grade is made by reducing rise and fall, the saving would be the consumption on the 8-percent grade (AB) minus the consumption on the 4-percent grade (AH). The consumption on AB (100 feet) was previously determined to be 0.001491 gallon. By using the rate of consumption shown in figure 23 for the 4-percent grade, the fuel consumed on AH (100 feet) was determined to be 0.000992 gallon. A saving of 0.000499 gallon or 33.4 percent resulted.

It is seen in table 5, that Method II always results in the largest saving. A reduction in grade by Method I appears to result in appreciable savings for grades in excess of 6 percent. However, grades of 6 percent or under must be reduced by Method II if any substantial saving is to be realized. It is emphasized that the savings shown in table 5 are based on the fuel characteristics of one passenger car, and that they could be materially different for other vehicles.

The differences between the two methods of grade reduction are very clearly shown in figure 32. The savings are those shown in table 5 for a sustained speed of 50 miles per hour. Except for the reduction of an 8- to a 6-percent grade, Method I is shown to be much inferior to Method II. Very little is gained by reducing grades of 6, 4, or 3 percent by Method I, or by reducing grades of 4 and 3 percent by either method. It can be readily seen that reducing grades per se may not result in appreciable savings in fuel consumption.

Fuel computations

The 21.0-mile section of U. S. 40 between Frederick and Hagerstown, Md., was selected for checking various methods that can be used to measure and compute fuel consumption, because the lengths of steep grades constituted a sizable portion of the total length. This section of highway had a rate of rise and fall of 3.7, the highest of any test route studied. About 29 percent of its length was on grades of 5 percent or more, and about 15 percent on grades of 7 percent or greater.

[4] *Braking distances of vehicles from high speeds, and tests of friction coefficients*, by O. K. Normann. PUBLIC ROADS, vol. 27, No. 8, June 1953.

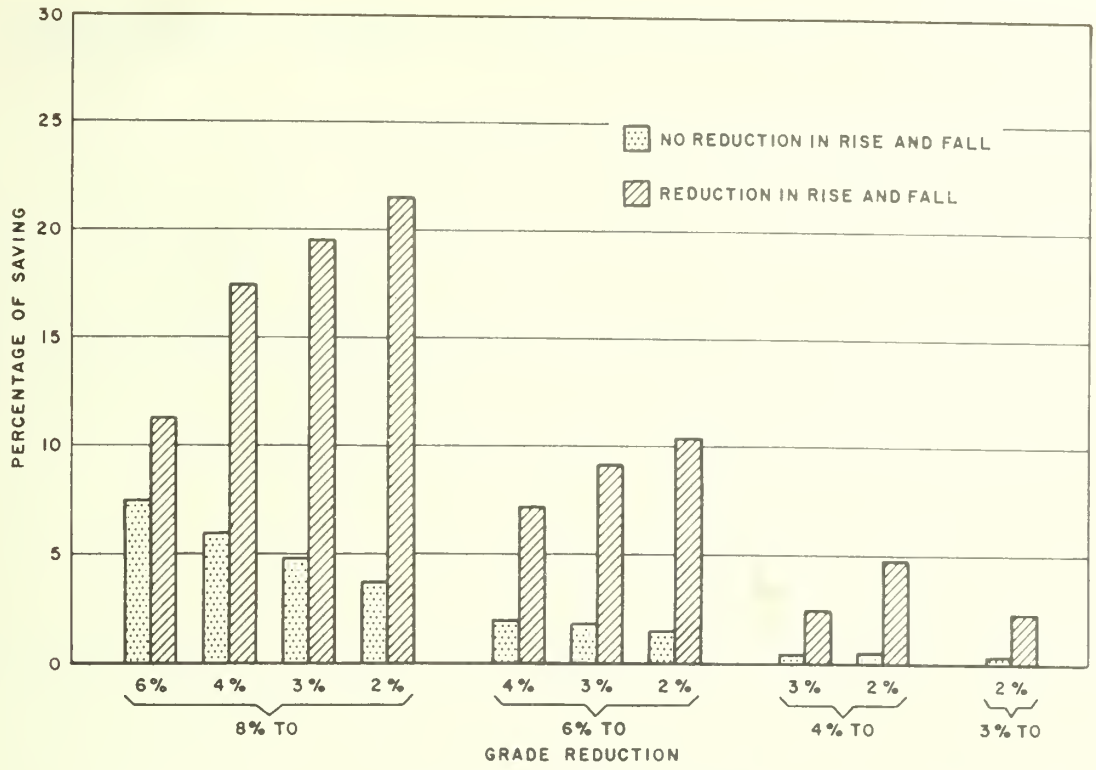

Figure 32.—Savings in fuel consumption determined by two methods of grade reduction for a sustained speed of 50 m. p. h.

The fuel consumption in gallons, determined by the various methods for an attempted speed of 50 miles per hour, is shown in table 6. Fuel was measured with a burette in one test, and with the fuelmeter in three tests. The fuel consumption was computed by two methods that use individual grades, and by two methods that use the rate of rise and fall, which have been called the composite or average grade by other investigators.

The values in the column headed "Individual grade method" are the summation of fuel consumption computed for each individual grade in the section. This method required 198 computations using the rates of fuel consumption shown in figure 23.

The "Grade classification method" is a simplified version of the method just discussed. The individual grades were grouped in four classes: 0 to 3 percent, 3 to 5 percent, 5 to 7 percent, and 7 to 9 percent. The total length in each class was then multiplied by the rate of fuel consumption in gallons per mile, obtained from figure 23 for the midpoint of the particular grade class. This method is not quite so laborious as the previous one and gave almost identical results.

The "Rise and fall method" required only one computation for a given section. The first column under this method contains values that were computed with the fuel consumption rates shown in figure 30 for various rates of rise and fall. The values in the second column headed "Individual grade relation" were based on the rates for individual grades shown in figure 23.

The fuel measured with the burette was used as a common base for comparative purposes. The percentage of variations from the burette measurement, shown in table 6, indicates that all methods gave results which were within reasonable limits of error. The much simpler "Rise and fall method" appears to be as good as, or better than, the two methods which require a solution for each individual grade. The results obtained with the fuelmeter also did not vary appreciably from those measured with the burette.

Public Utility Relocation Incident to Highway Improvement

President Eisenhower on April 5, 1955, transmitted to Congress a report on the problems posed by relocation of public utilities made necessary by highway improvements. The report was prepared by the Bureau of Public Roads, in cooperation with the State highway departments and public utilities, at the direction of Congress, in section 11 of the Federal-Aid Highway Act of 1954.

The report, entitled *Public Utility Relocation Incident to Highway Improvement*, has been published as House Document No. 127, 84th Congress, 1st Session, and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

Traditionally, public utilities have established themselves along, and within, public highway rights-of-way. In general, the status of these installations has been covered by State constitutional or statutory authority. As a rule, where highway improvements require it, the public utility facilities are removed from one location within the highway right-of-way to another at the expense of the utilities themselves. As highway improvement and modernization increased, the utilities have claimed that these relocation costs imposed a greater and greater burden.

In its study of this problem the Bureau of Public Roads investigated both the cost of public utility relocation and the legal relationships which may affect the distribution of the costs.

The State highway departments reported that the total dollar value of all highway projects completed in the survey year 1953 was approximately $1.7 billion and involved 10,245 highway projects, aggregating 40,027 miles in length. The public utilities which cooperated in the study reported that they could identify 5,422 utility relocations in connection with 3,836 of these highway projects. The dollar value of this construction amounted to about $1.1 billion and involved nearly 14,000 miles of highway.

The utilities reported relocation costs for the year amounting to $35.5 million. More than 80 percent of this cost ($29.1 million) involved utilities located within the highway right-of-way. The remaining 20 percent ($6.4 million) was the cost of moving utilities located on their own private rights-of-way for much of which they were reimbursed in the same manner as any other property owner. Total costs were divided almost equally between projects in urban and rural areas—$15.2 million in urban areas and $13.9 million in rural areas. Of the reported $35.5 million relocation cost, nearly 90 percent was incurred in connection with the Federal-aid systems, the remainder on State highways outside the Federal-aid systems.

Among utility types, electric and power, and telephone together accounted for nearly 70 percent of the reported $35.5 million relocation cost, gas accounted for almost 14 percent, and water for some 11 percent. Some of these costs were reimbursable and agencies at different levels of government were involved in making payments. The bulk of the reimbursements were met by the State highway departments. Of a total reimbursement of more than $8.4 million, 86.5 percent, or nearly $7.3 million, was so returned to the utilities.

Data which the Bureau of Public Roads obtained from the State highway departments with respect to reimbursement for public utility relocation costs indicated that the total of State reimbursements for such costs was nearly $16 million. It is evident that the State highway department figures include many public utility highway relocation projects which were not included in the data reported by the public utilities. This is notably true of publicly owned utilities.

Precast Concrete Bridge: A Motion Picture

The Bureau of Public Roads recently produced and released a new motion picture, *A Record of Casting and Placement of a Precast Concrete Bridge*. The one-reel, 16-mm. sound and color film has a running time of 18 minutes.

The film illustrates, with scenes at an actual bridge site and with animated drawings, a method of bridge building which is growing in usage in this country. The nature of the forms for the precast concrete beams, deck slabs, and curb sections, and their use in the central casting yard, are demonstrated in detail. Driving of piles and construction of bent-caps at the bridge site, the placement of the precast units, and the final operations involved in completing the three-span structure are shown step by step.

The film may be borrowed by any responsible organization, without charge except for the nominal shipping costs, by writing to Visual Education, Bureau of Public Roads, Washington 25, D. C. The number of available prints is limited, so several alternate dates should be proposed in requesting loan of the picture. Loans can be made only for short periods of time.

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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Work of the Public Roads Administration:
1941, 15 cents. 1948, 20 cents.
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Public Roads Administration Annual Reports:
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(*Free from Bureau of Public Roads*)
Annual Reports of the Bureau of Public Roads:
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1951, 35 cents. 1953, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
Construction of Private Driveways, No. 272MP (1937). 15 cents.
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Highway Capacity Manual (1950). $1.00.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
1945, 35 cents. 1948, 65 cents. 1951, 60 cents.
1946, 50 cents. 1949, 55 cents. 1952, 75 cents.
1947, 45 cents. 1950 (out of print). 1953, $1.00.
Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (1950). Out of print.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.
Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Model Traffic Ordinance (revised 1953). 20 cents.

PUBLICATIONS (Continued)

Needs of the Highway Systems, 1955 84, House Document No. 120 (1955). 15 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.
Standard Plans for Highway Bridge Superstructures (1953). $1.25.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 21–27.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1955

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$6,590	$17,420	$9,502	414.3	$4,881	$2,591	48.1	$44,670	$22,626	752.8	$66,971	$34,719	1,215.2
Arizona	1,879	3,837	2,900	102.1	3,324	2,345	53.2	8,641	6,324	162.7	15,802	11,569	318.0
Arkansas	7,344	7,764	5,263	334.4	4,379	2,340	48.1	18,850	9,464	504.8	32,993	17,067	887.3
California	1,654	18,093	9,276	250.2	14,950	7,430	14.1	122,651	60,290	309.6	155,694	76,996	573.9
Colorado	12,815	4,226	2,387	69.7	1,043	271	.2	19,060	10,498	220.4	24,329	13,156	290.3
Connecticut	15,224	1,534	771	6.7	1,501	733	2.5	8,456	4,159	10.8	11,491	5,663	20.0
Delaware	3,916	888	454	15.4	2,056	1,034	4.9	6,419	3,552	25.7	9,363	5,040	46.0
Florida	4,608	25,684	13,366	378.7	5,935	2,981	61.1	25,185	12,967	288.9	56,804	29,314	728.7
Georgia	14,196	16,298	8,224	353.6	6,018	2,480	26.5	51,449	24,470	920.0	73,765	35,174	1,300.1
Idaho	1,996	6,335	4,036	94.2	5,085	3,337	112.8	15,277	9,735	242.0	26,697	17,108	449.0
Illinois	7,864	36,536	20,161	432.0	16,784	9,207	41.7	92,483	48,265	763.4	145,803	77,633	1,237.1
Indiana	11,409	24,904	13,426	117.4	14,696	7,375	76.3	53,268	27,858	177.3	92,868	48,729	371.0
Iowa	5,010	21,012	11,273	878.1	4,250	2,379	66.4	26,784	14,622	912.5	52,046	28,274	1,857.0
Kansas	7,733	10,543	5,455	986.1	4,933	2,494	132.4	23,857	11,897	883.6	39,333	19,846	2,002.1
Kentucky	5,634	13,196	6,827	359.6	8,622	4,710	70.6	32,435	16,285	445.0	54,253	27,822	875.2
Louisiana	5,185	12,161	6,080	107.1	8,493	4,247	135.0	39,021	18,386	378.3	59,675	28,713	620.4
Maine	2,009	10,714	5,649	89.7	288	146	2.4	16,056	8,259	129.3	27,058	14,054	221.4
Maryland	5,204	20,087	10,580	60.9	6,417	2,932	15.4	12,366	6,575	97.0	38,870	20,087	173.3
Massachusetts	13,953	6,214	3,097	23.2	3,687	1,831	6.6	49,040	22,931	51.5	58,941	27,859	81.3
Michigan	3,347	55,995	28,864	783.3	13,107	6,674	128.7	42,707	21,589	411.8	111,809	57,127	1,323.8
Minnesota	6,695	9,973	5,423	466.8	12,776	6,591	999.0	31,054	16,217	1,022.7	53,803	28,231	2,488.5
Mississippi	2,259	11,314	5,414	476.7	8,590	4,725	135.4	28,432	14,658	698.4	48,336	24,797	1,310.5
Missouri	8,554	18,270	9,315	976.7	3,780	2,092	18.8	80,551	41,858	1,456.4	102,601	53,265	2,451.9
Montana	9,204	14,535	9,353	286.8	3,967	2,405	148.6	24,910	15,535	515.4	43,412	27,293	950.8
Nebraska	6,426	22,662	11,746	1,006.5	9,260	4,337	161.6	27,534	15,375	699.4	59,456	31,458	1,867.5
Nevada	9,308	2,047	1,711	68.3	154	129	4.6	7,924	6,631	139.4	10,125	8,471	212.3
New Hampshire	3,690	4,222	2,111	27.0	9	8		8,786	4,463	49.8	13,017	6,582	76.8
New Jersey	13,072	17,672	8,529	68.5	4,859	2,409	13.4	21,369	9,661	48.2	43,900	20,599	130.1
New Mexico	5,147	3,094	1,985	72.9	2,575	1,667	89.7	13,330	8,453	217.7	18,999	12,105	380.3
New York	30,161	28,670	15,321	96.9	50,709	24,459	91.0	200,644	94,298	263.7	280,023	134,078	451.6
North Carolina	6,767	23,109	11,367	424.4	6,455	3,273	89.3	41,117	20,103	575.6	70,681	34,743	1,089.3
North Dakota	3,326	7,169	3,622	1,070.6	10,874	5,384	782.9	8,787	4,557	485.9	26,830	13,563	2,339.4
Ohio	12,105	54,585	27,455	125.2	5,251	2,656	30.0	64,728	30,774	127.8	124,564	60,885	283.0
Oklahoma	14,677	5,046	2,615	182.9	17,130	8,577	234.5	25,346	13,415	365.0	47,522	24,607	782.4
Oregon	2,929	7,925	4,723	102.4	2,564	1,518	24.1	16,206	10,020	242.6	26,695	16,261	369.1
Pennsylvania	7,830	56,081	29,327	116.2	17,519	8,954	42.9	97,890	48,486	234.7	171,490	86,767	393.8
Rhode Island	3,047	5,739	2,869	13.1	185	93		11,338	5,682	36.9	17,262	8,644	50.0
South Carolina	6,537	15,710	8,357	416.1	1,273	749	5.3	18,697	9,889	292.5	35,680	18,995	713.9
South Dakota	3,214	16,669	9,643	915.6	2,228	1,298	107.9	8,814	5,051	401.0	27,711	15,992	1,424.5
Tennessee	11,940	16,638	8,274	306.2	5,244	2,622	38.0	38,515	17,448	590.7	60,397	28,344	934.9
Texas	18,104	19,261	10,512	488.2	13,206	6,632	114.8	83,314	43,944	1,225.8	115,781	61,088	1,828.8
Utah	753	7,912	5,999	123.4	1,740	1,225	33.8	10,035	7,626	169.4	19,687	14,850	326.6
Vermont	3,394	917	463	14.3	282	230	.4	8,823	4,530	86.7	10,022	5,223	101.4
Virginia	12,778	11,022	5,534	253.9	5,280	2,468	38.9	22,630	11,281	317.0	38,932	19,283	609.8
Washington	3,036	17,722	9,816	190.9	3,392	1,985	134.3	23,241	12,277	168.0	44,355	24,078	493.2
West Virginia	10,611	10,471	5,335	54.4	4,764	2,394	40.0	13,804	6,947	49.0	29,039	14,676	143.4
Wisconsin	6,937	16,657	8,275	328.9	10,563	5,276	67.8	39,570	19,827	542.0	66,790	33,378	938.7
Wyoming	989	4,303	2,788	89.8	3,168	2,069	65.6	13,505	8,670	292.6	20,976	13,527	448.0
Hawaii	3,137	2,426	1,195	4.4	2,377	1,183	.8	5,478	2,452	12.7	10,281	4,830	17.9
District of Columbia	9,346	5,229	2,612	7.1	275	137	.6	12,752	6,038	2.9	18,252	8,787	10.6
Puerto Rico	8,482	2,633	1,273	15.3	4,676	2,284	16.8	13,274	5,925	45.7	20,583	9,482	77.8
TOTAL	377,96[illegible]	795,120	[illegible]00,623	14,647.1	345,574	177,366	4,577.8	1,731,073	882,843	19,063.0	2,831,767	1,460,832	38,287.9

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U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: Traffic studies of the Maine and Pennsylvania Turnpikes
(Pennsylvania Turnpike between Gettysburg Pike and Harrisburg-West Shore interchanges)

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 28, No. 10 *October 195*

Published Bimonthly

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The printing of this publication has been approved by the Director of the Bureau of the Budget, March 17, 1955.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

Contents of this publication may be reprinted. Mention of source is requested.

Pennsylvania Turnpike Traffic Analysis

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by DANIEL O'FLAHERTY
Head, Traffic and Travel Studies Unit

Information regarding turnpike usage was collected at roadside-interview stations on the highways east of Harrisburg and west of Philadelphia during June, July, and August of 1950 before construction of the eastern extension of the Pennsylvania Turnpike, and on the same highways and the turnpike in 1952 after its construction.

Traffic diversion to the toll road was much greater for passenger cars than for commercial vehicles, except in the mountainous areas where the type of vehicle or purpose of trip made little difference.

It was found that a much greater increase in traffic in the vicinity and general direction of the turnpike resulted after its opening than elsewhere in the State, indicating that this facility generated a considerable amount of new traffic.

The travel-time ratio and time saved or lost were found to be good measures for determining the amount of traffic diverted from adjacent highways. Traffic used the toll road if time could be saved, even though the distance was increased; hence the greater the time saving, the greater the usage. Only a small proportion of traffic losing both time and distance used the toll road.

The toll-road usage curves differ from similar curves established for free limited-access highways in metropolitan areas. It was found that the smaller the time or distance saved, the greater the difference between such curves. Traffic diverted to the Pennsylvania Turnpike was between two-thirds and three-fourths of that which would have been assigned from curves developed for free limited-access highways.

Vacation trips as a whole accounted for greater usage of the toll road than trips for other purposes—presumably because there were a few very short trips and many long trips with vacations as the purpose.

Vehicles traveling between Philadelphia and Pittsburgh used the turnpike to a very large extent regardless of purpose of trip and type of vehicle due to the large saving of time for all vehicles and of fuel as well for commercial vehicles.

ONE of the most difficult problems facing highway planning engineers is estimating the traffic which would use improved highways of various classes. This is important from the points of view of economic justification, priorities, and design features. With present emphasis on the completion of the National System of Interstate Highways the study of the usage of controlled-access highways, such as will constitute a large part of that system, assumes special importance.

When a high-type facility is constructed, there is a large diversion of traffic from roads immediately parallel and a lesser diversion from more distant roads. In addition, there is an increase in highway traffic throughout the area which is called traffic generation. A part of this increase is composed of trips diverted from public carriers, and an important part is composed of new trips which would not have been made if the improved facility had not been available. Furthermore, there is the long-range traffic growth or trend which follows generally the basic economic trend and takes place on highways of all classes throughout the country.

In estimating traffic usage of a proposed facility, therefore, the engineer must concern himself with these three factors: Diversion, generation, and long-range trend. To do this successfully, empirical data, properly analyzed, are needed. The Bureau of Public Roads, with the cooperation of the various State highway departments and other agencies, is endeavoring to supply these data and analyses to the extent practicable.

Other Traffic Studies Made

With regard to controlled-access facilities such as are contemplated in the Interstate system standards, several studies have been made and reported for highways in urban areas,[1] but very few for those in rural areas. There are few controlled-access free roads extending for long distances through rural areas and these have been opened to traffic only recently. There are, however, several toll roads or turnpikes built approximately to Interstate system standards, which have been in operation for a number of years and others which are being opened from time to time. These facilities offer good opportunities for study, though the deterring effect on traffic of the toll charges offers a complicating factor which must be evaluated in applying the results to free roads.

For a number of years the Bureau of Public Roads has cooperated with several States through the planning survey divisions of the highway departments in studies of toll-road traffic in relation to that on adjacent highways. The toll authorities have supplied at regular intervals information on traffic using the turnpikes. Such information, in addition to regular reports from the State highway departments concerning traffic volumes throughout the country, has given a broad picture of the changes in traffic flow which result from the construction of modern toll roads. This in itself, however, is not enough to permit accurate traffic estimates for any particular project under study.

Conclusions which can be drawn from this type of information are somewhat clouded by the fact that there is no differentiation between passenger car and truck traffic, or between trips which could conveniently be made on the turnpike and those which are purely local. Relations determined from traffic counts alone would be expected to vary widely in different areas according to the composition and characteristics of traffic in the area. In order to establish more basic relations, intensive traffic studies were made on several of the turnpikes. In these studies, drivers were interviewed at stations on roads parallel to the turnpike route both before and after the opening of the turnpikes, and at all turnpike interchanges under study after its opening. Questions were asked concerning trip origins and destinations, trip purpose, and other matters pertinent to the study.

Such studies were made in Maine, New Hampshire, New Jersey, Pennsylvania (eastern and western extensions), and Ohio. The "after" study has not been conducted in Ohio, because only one short section has been completed. In New Jersey the data are being analyzed. The results of the Maine Turnpike study have been partially reported to the Highway Research Board and a further report is included in this issue.[2] The present article reports the results of the study of the eastern extension of the Pennsylvania Turnpike which was opened to traffic in November 1950.

[1] *Traffic assignment*, Highway Research Board, Wash., D. C., Bulletin 61, publication 246, 1952; also *Gulf Freeway traffic survey, Houston, Texas.* Texas Highway Department, 1951.

[2] *Proportionate use of Maine Turnpike by traffic through Portsmouth-Portland corridor*, by J C Carpenter. Proceedings of the Highway Research Board, vol. 32, 1953, pp. 452-463; also *Traffic usage of Maine Turnpike*, by Glenn E. Brokke. Pages 224-230 of this issue.

355212—55——1

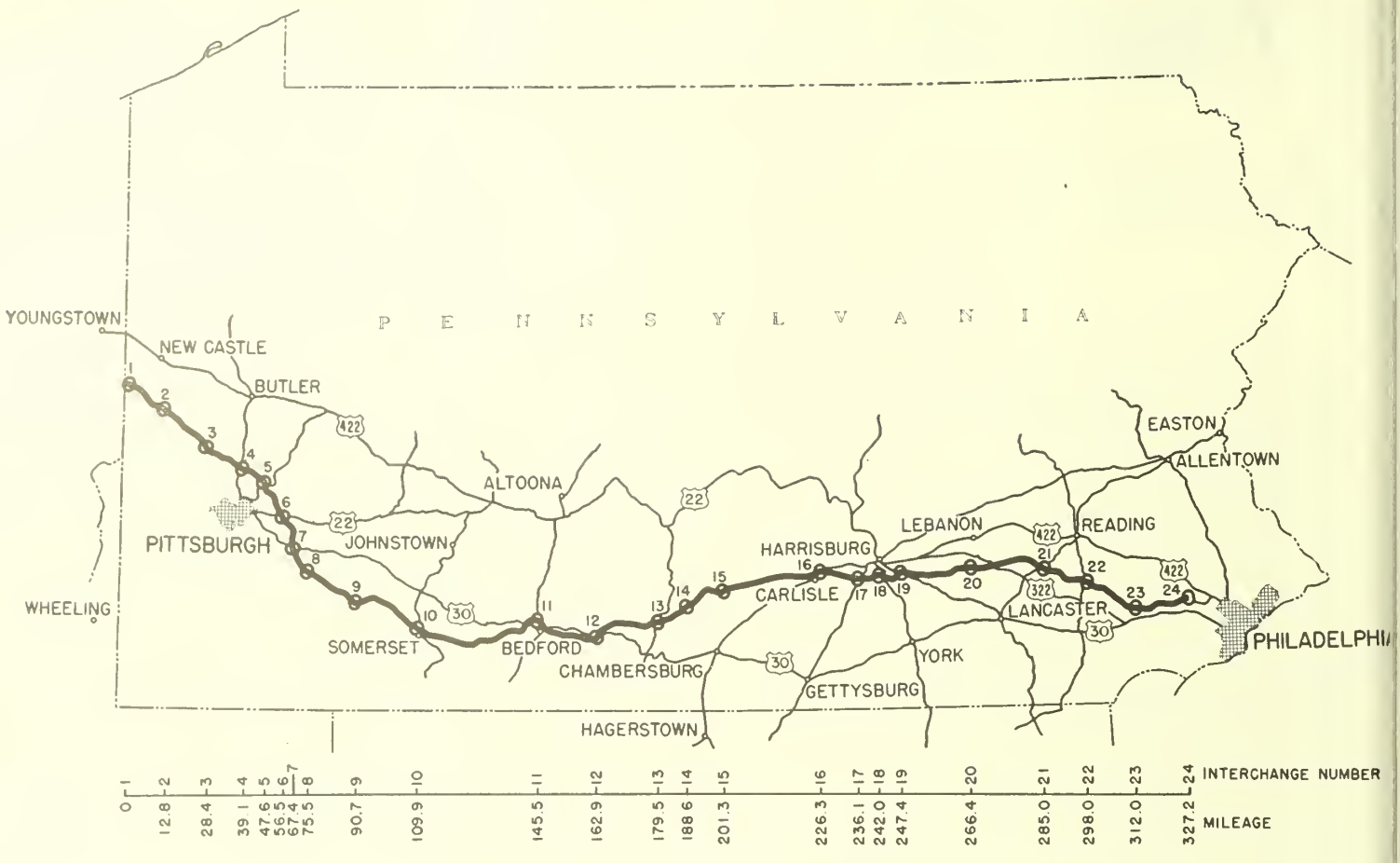


Figure 1.—Location of the Pennsylvania Turnpike.

Pennsylvania Turnpike Study

The location of the entire Pennsylvania Turnpike is shown in figure 1. The interchanges are indicated by the numbered circles, and mileages along the turnpike measured from the western terminus are shown at the bottom of the illustration.

The original section, which was opened in October 1940, extends from interchange 7 at Irwin (near Pittsburgh) to interchange 16 at Carlisle, a distance of 158.9 miles. A profile of this section of the turnpike and the principal parallel free routes (U. S. 30 and 11) appeared in a recent article in Public Roads.[3] The grades on the turnpike are much lighter and the total rise and fall much less than on the parallel free route. The rise and fall on the turnpike (original section) is 11,327 feet, compared to 26,257 feet on the free route. The comparatively low grades and saving in climb have much to do with the high usage of this section of the turnpike especially by trucks.

The eastern extension of the turnpike, which was opened in November 1950, extends from interchange 16 at Carlisle to interchange 24 at Valley Forge (near Philadelphia), a distance of 100.9 miles. It was on this section that the intensive traffic study was made. Origin and destination stations were operated in 1950 on one line just east of Harrisburg and on a second line about 20 miles west of Philadelphia before the turnpike was completed, and in 1952 after its construction. The location and types of survey stations are shown in figure 2, which is an enlargement of the study area.

There were 15 identical interview stations operated on the highways in the "before" and "after" studies made during June, July, and August of each of the 2 years. Additional interview stations were located at the turnpike interchanges and at 3 locations on the free highways in the after study. The stations were occupied on approximately the same dates in each year for the same number of hours. The duration of the operations was for at least a full 24-hour period on weekdays in June, July, or August. Additional 24-hour operations were made on Saturday and Sunday at a limited number of locations.

The topography along this section of the turnpike is generally level as compared with the rugged terrain through which the original section was built. There are four or more parallel routes in the eastern section from which traffic is diverted.

Since the major parallel routes are high-type and well-maintained highways, the big traffic obstacle in this area is the large number of towns and cities located on the regular routes. Such places cause traffic to slow down even when highways are adequate. During peak hours the highways in and near these municipalities are congested, and delays at such times may be considerable.

During the survey the routes connecting Valley Forge with Philadelphia were inadequate to serve properly the volume of traffi delivered by the turnpike. There was litt choice between the routes from Valley Forg and from Paoli on U. S. 30 to Philadelphi Both offered traffic resistance, especially durin peak traffic periods. An expressway is unde construction between Valley Forge and Phila delphia which will relieve this situation an may cause greater turnpike usage than o curred in 1952.

The turnpike has a minimum right-of-wa width of 200 feet, and has four 12-foot traff lanes divided by a 10-foot median strip i addition to acceleration and deceleration lan at interchanges. There are 10-foot pave shoulders. The maximum grade is 3 percen the maximum curve is 6 degrees, and the mini mum sight distance is 1,000 feet.

The average distance between interchange in the 327.2 miles between the Ohio State lin and Valley Forge is 14.2 miles. The shortes section, 5.4 miles, crosses the Susquehann River south of Harrisburg between inter changes 18 and 19, and adjoins the longes section on the eastern extension which is 1 miles in length and connects interchanges 1 and 20.

The speed limit for passenger vehicles o the turnpike is 60 miles per hour west o Breezewood, on the original section, and 7 miles per hour east of that point including th eastern extension. It is 45 and 50 miles pe hour, respectively, for all other vehicles ex cept where lower speeds are posted, such as i tunnels and at toll-gate approaches.

[3] *Operating characteristics of a passenger car on selected routes*, by Carl C. Saal. Public Roads, vol. 28, No. 9, Aug. 1955, p. 181.

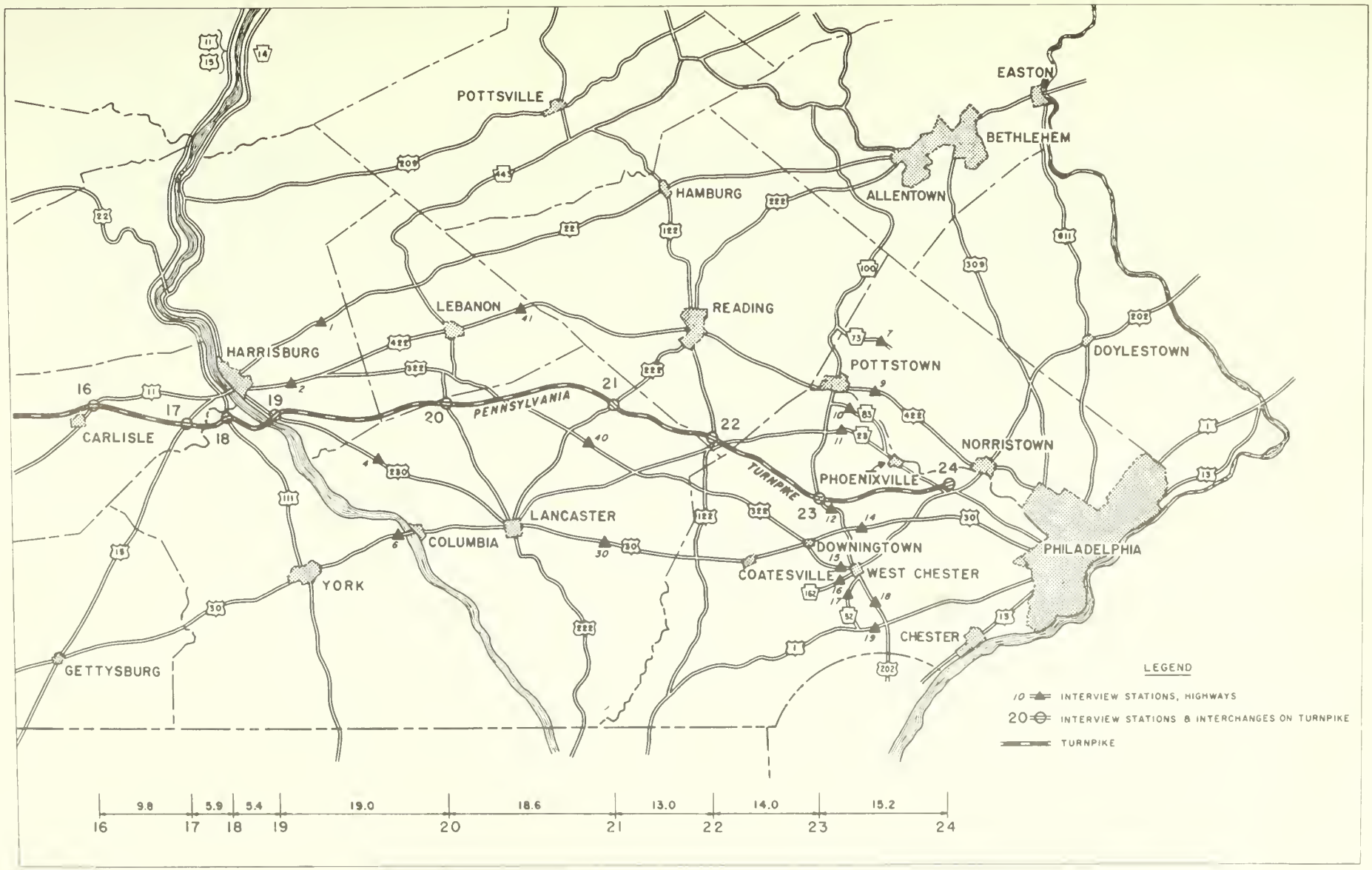


Figure 2.—Map of study area of the eastern extension of the Pennsylvania Turnpike showing the turnpike, other highways, and locations of roadside interview stations.

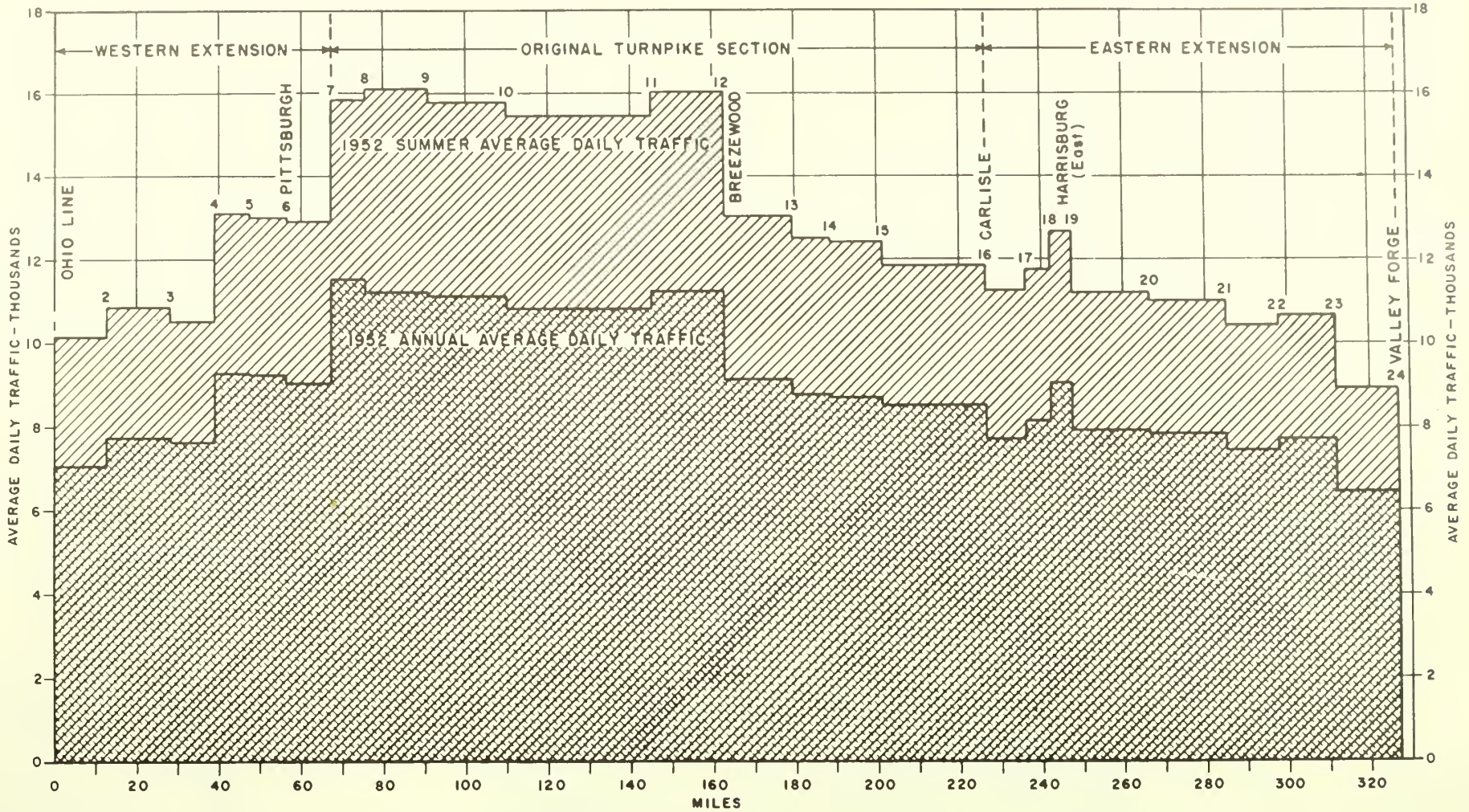


Figure 3.—Annual and summer average daily traffic between interchanges in 1952.

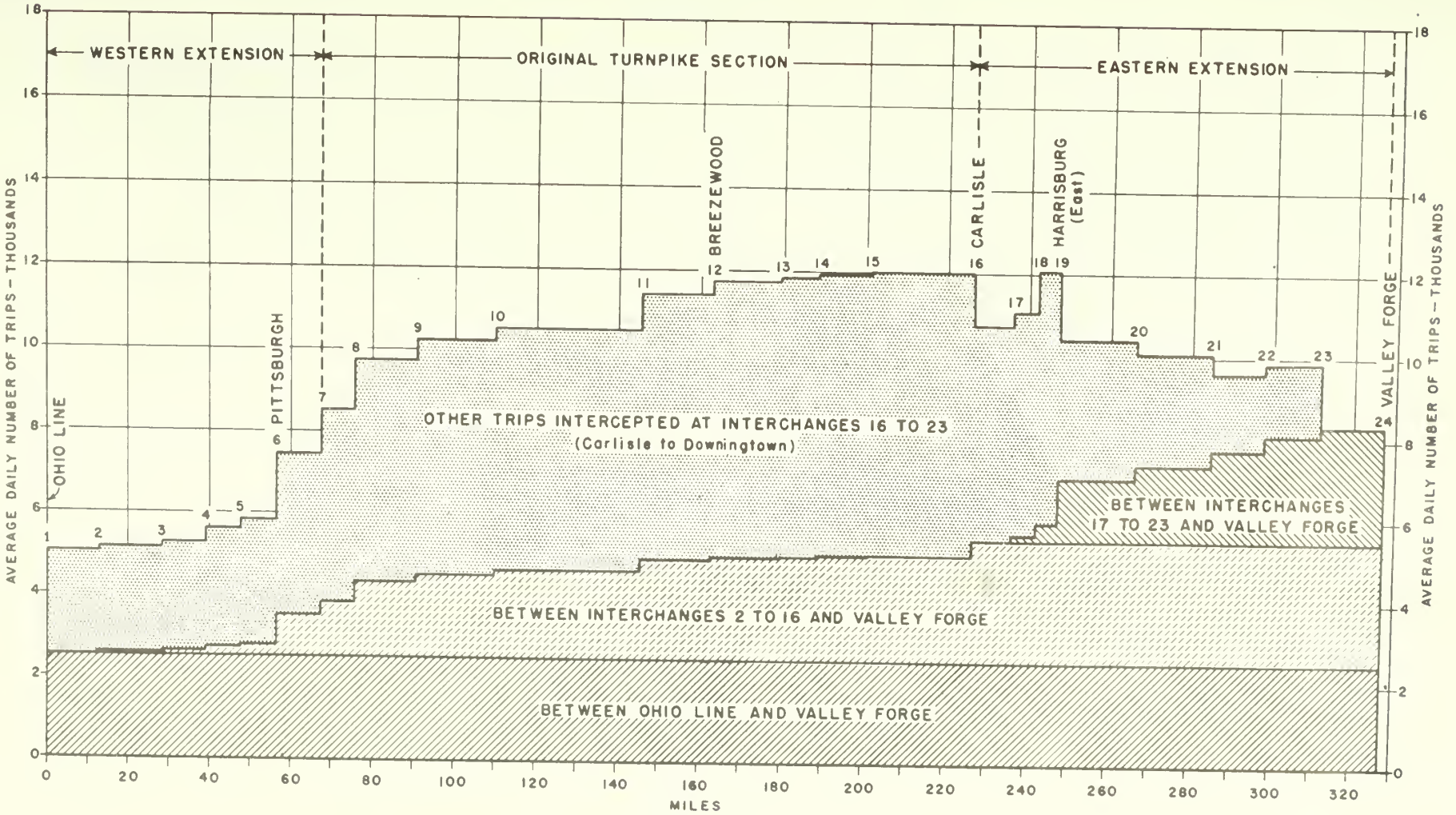


Figure 4.—Summer average weekday traffic between interchanges sampled in 1952 survey.

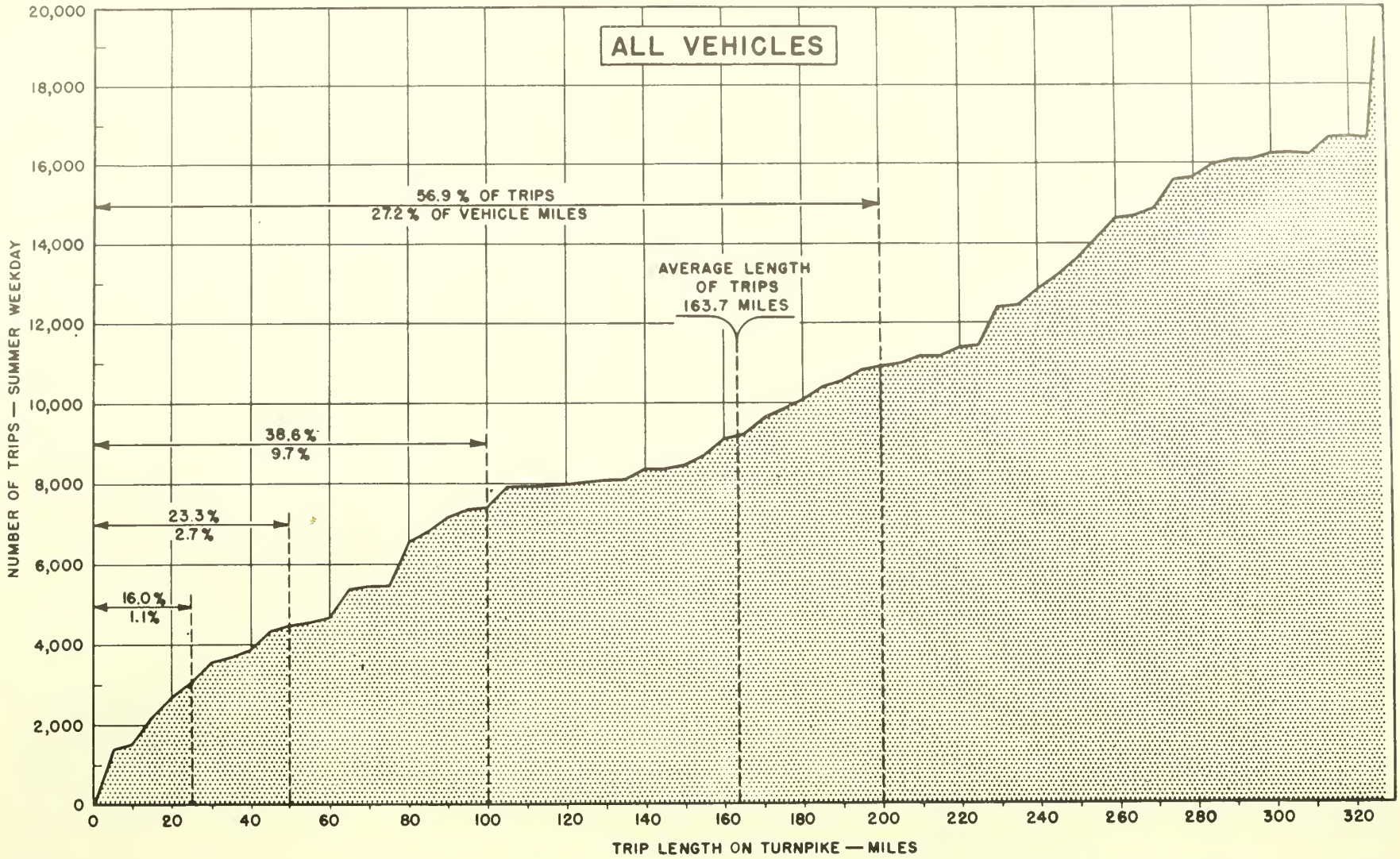


Figure 5.—Cumulative number of weekday trips related to miles of travel on the turnpike, and the number of such trips related to total trips and vehicle-miles.

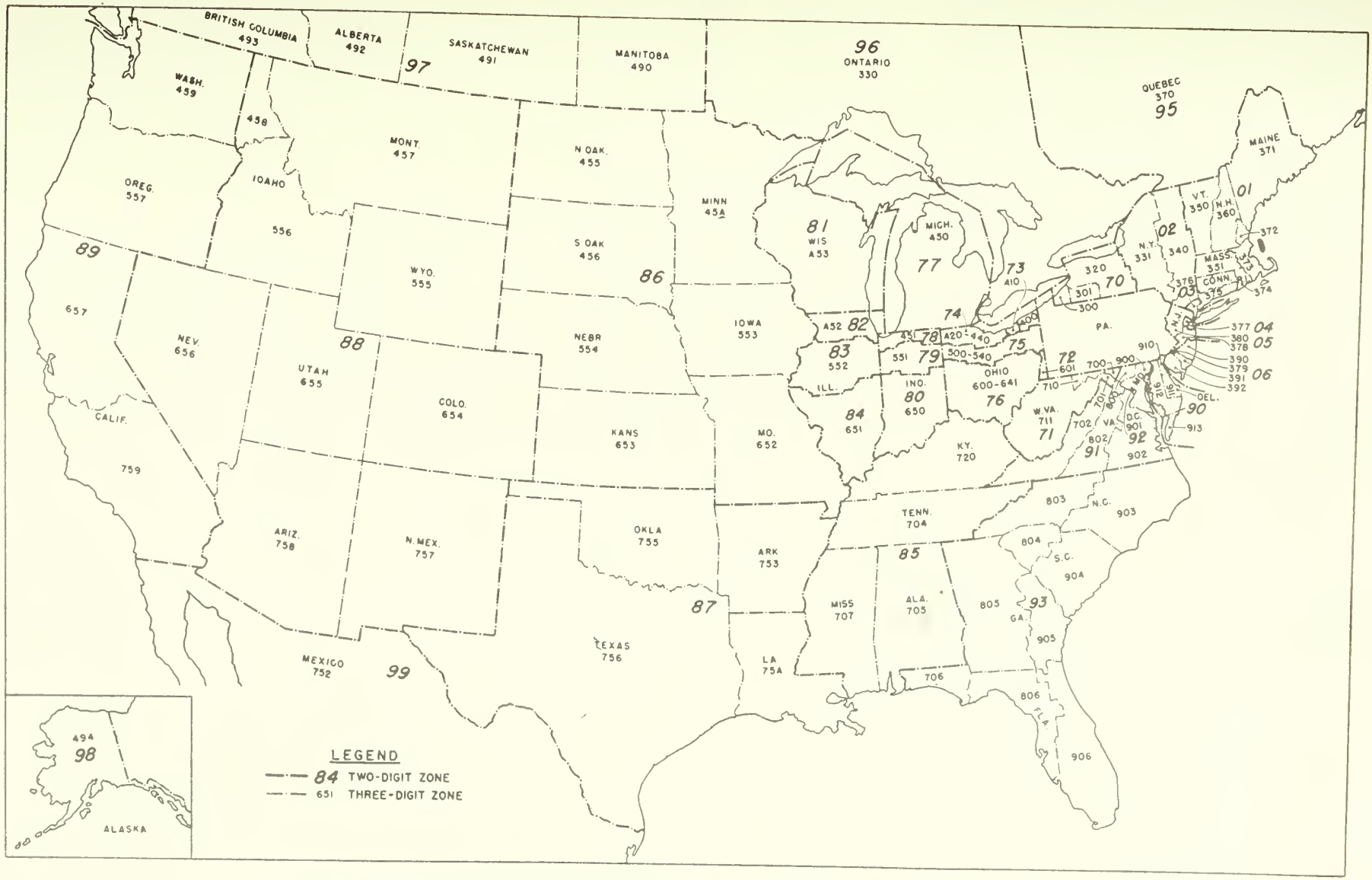


Figure 6.—Map of the United States indicating 2- and 3-digit zones, except for Pennsylvania.

Table 1.—Number of trips between interchanges on the Pennsylvania Turnpike intercepted on the eastern extension during an average summer weekday in 1952

Interchange	Carlisle (16)	Gettysburg Pike (17)	Harrisburg-West Shore (18)	Harrisburg East (19)	Lebanon-Lancaster (20)	Reading (21)	Morgantown (22)	Downingtown (23)	Valley Forge (24)	Total
1 Ohio State line	850	118	224	410	184	291	58	398	2,509	5,042
2 Beaver Valley	23	2	9	21	3	3		14	41	116
3 Perry Highway	18	1	8	10	7	5	1	18	56	124
4 Butler Valley	54	13	48	34	11	16	4	36	113	329
5 Allegheny Valley	33	5	10	27	15	13	2	35	60	200
6 Pittsburgh	193	45	101	198	57	98	30	174	746	1,642
7 Irwin	325	56	73	119	31	43	14	91	314	1,066
8 New Stanton	226	48	74	143	52	59	16	75	526	1,219
9 Donegal	122	26	39	57	15	14	7	35	171	486
10 Somerset	70	9	24	28	15	26	1	32	123	328
11 Bedford	198	22	62	101	22	57	13	78	314	867
12 Breezewood	109	9	7	34	10	29	3	34	73	308
13 Fort Littleton	33	6	12	12	3	6	1	7	27	107
14 Willow Hill	26	6	13	18	1		2	4	25	95
15 Blue Mountain	27	5	2	12	1			19	17	83
16 Carlisle		31	87	281	61	86	15	65	373	999
17 Gettysburg Pike			96	400	54	44	7	17	145	763
18 Harrisburg-West Shore				1,401	78	52	14	61	307	1,913
19 Harrisburg East					140	147	51	233	1,072	1,643
20 Lebanon-Lancaster						36	41	63	355	495
21 Reading							67	86	363	516
22 Morgantown								194	394	588
23 Downingtown									233	233
Total	[1] 2,307	402	889	3,306	760	1,025	347	1,769	8,357	19,162

[1] Trips off or on at Carlisle and traveling to or from the west did not use the eastern extension.

Traffic Between Interchanges

The average daily traffic volume on all sections of the turnpike, in the summer and throughout the year, is shown in figure 3. Saturdays and Sundays as well as weekdays are included. The volume on the original section is greater than that on either the eastern or western extension. The summer traffic is about 29 percent greater than the average annual traffic.

The trips intercepted on the eastern extension of the turnpike were analyzed on the basis of point of entry and point of exit to determine their travel on the turnpike. The results are shown in figure 4 for an average summer weekday.

The hatching at the bottom of the chart indicates the trips over the whole length of the Pennsylvania Turnpike between the Ohio State line and Valley Forge. The next group includes vehicles on trips that used the full length of the eastern extension extending from Valley Forge to Carlisle or farther west. The third group of trips are those intercepted at Valley Forge which are shorter than the length of the eastern extension. This accounts for the total trips at Valley Forge. The remaining trips are those which do not extend to Valley Forge, but enter or leave the turnpike between Carlisle and Downingtown. The latter group includes trips entirely on the eastern extension as well as those on all or portions of the turnpike west of Carlisle.

The apparent decrease in turnpike use west of the eastern extension is due to the omission in figure 4 of all trips which enter and leave the turnpike between the Ohio State line and a point west of Carlisle. Such trips were not sampled in the eastern extension study.

The length of that portion of a trip which lies between the termini of a controlled-access facility is important in determining diversion to the facility. It is of particular interest to note some of the results found in this study regarding the length of that portion of the trips found on the turnpike itself. There were 2,509 vehicles, making up approximately 25 percent of the vehicle-mileage on the eastern extension of the Pennsylvania Turnpike, that used the entire 327 miles in traveling between the Ohio line and Valley Forge. Another 2,979 vehicles traveled at least the 101 miles of the eastern extension between Valley Forge and Carlisle. This number of trips adds to 55 percent of the vehicle-mileage on the eastern extension during the summer months. That is, during the period of the survey, more than one-half of the vehicle-miles of travel on the eastern extension resulted from through trips on it (see fig. 4 and table 1). Almost two-thirds of the vehicles intercepted at Valley Forge traversed the whole length of the eastern extension.

There were 19,162 trips intercepted in the 1952 study on the eastern extension. Of this total, 2,307 vehicles were off or on at Carlisle traveling to or from the west and did not use the eastern extension.

Table 2 shows the percentages of the turnpike trips and of the vehicle-mileage by these trips, on different lengths of the turnpike. For example, 8.0 percent of the turnpike trips were less than 10 miles in length, and accounted for only 0.3 percent of the turnpike vehicle-mileage. All but 38.6 percent of the trips were for distances greater than 100 mile and all but 9.7 percent of the turnpike vehicle mileage was due to these long trips. Thi information is presented graphically in figure 5

The average length of the turnpike portio of the trip was 163.7 miles which is, of course less than the average length of trip from origi to destination. This figure is somewhat mis leading, however, since stations were operate only on the eastern extension and short trip between interchanges on the other section were not included in the sample, whereas all c the through trips were included.

The average trip length determined fror tickets for all types of vehicles for the whol turnpike was 98 miles for the year 1952 an 105 miles for the period of the survey. If onl the mileage on a particular section is cor sidered, the annual average length of trips wa as follows: Eastern extension, 55 miles original section, 97 miles; and the wester extension, 39 miles.

Traffic Diversion and Generation

As previously mentioned, the traffic tha uses a new facility is composed of both diverte and generated traffic. It is difficult to sepa rate the amount diverted from the amoun generated.

Diverted traffic is composed of vehicle traveling over existing roads before the cor struction of a new facility, and then transfer ring to the new route after its completion When there is a saving in time, driver generally transfer to a high-type free facility Fewer of such trips are diverted to a toll roac It is desirable to determine what proportion c eligible or potential trips between particula zones actually use the toll road and whethe time or distance is saved.

The term "generated traffic" as used b highway engineers in various areas of th country often has different meanings. Th American Association of State Highwa Officials has defined the term in one of it publications.[4] As used in this article, how ever, generated traffic refers to the amount c traffic using a new road, plus that on the ol roads in the corridor, less the amount tha would have been expected on the old highway had not the new facility been constructed That is, additional trips made over and abov those diverted from nearby roads to the ne facility—generally those which were not mad

[4] *A policy on geometric design of rural highways.* America Association of State Highway Officials, Wash., D. C., 195 p. 66.

Table 2.—Number of trips and vehicle-miles of travel by turnpike trip-length groups

Trip length, in miles	Trips for all vehicles				Vehicle-miles of travel			
	Total for group		Cumulative total		Total for group		Cumulative total	
	Number	Percent of total	Number	Percent of total	Number	Percent of total	Number	Percent of total
Under 10	1,528	8.0	1,528	8.0	8,436	0.3	8,436	0.3
10–25	1,543	8.1	3,071	16.1	24,851	.8	33,287	1.1
25–50	1,389	7.2	4,460	23.3	50,929	1.6	84,216	2.7
50–100	2,935	15.3	7,395	38.6	221,631	7.0	305,847	9.7
100–200	3,505	18.3	10,900	56.9	548,440	17.5	854,287	27.2
200–300	5,314	27.7	16,214	84.6	1,325,247	42.3	2,179,534	69.5
300–327	2,948	15.4	19,162	100.0	958,011	30.5	3,137,545	100.0

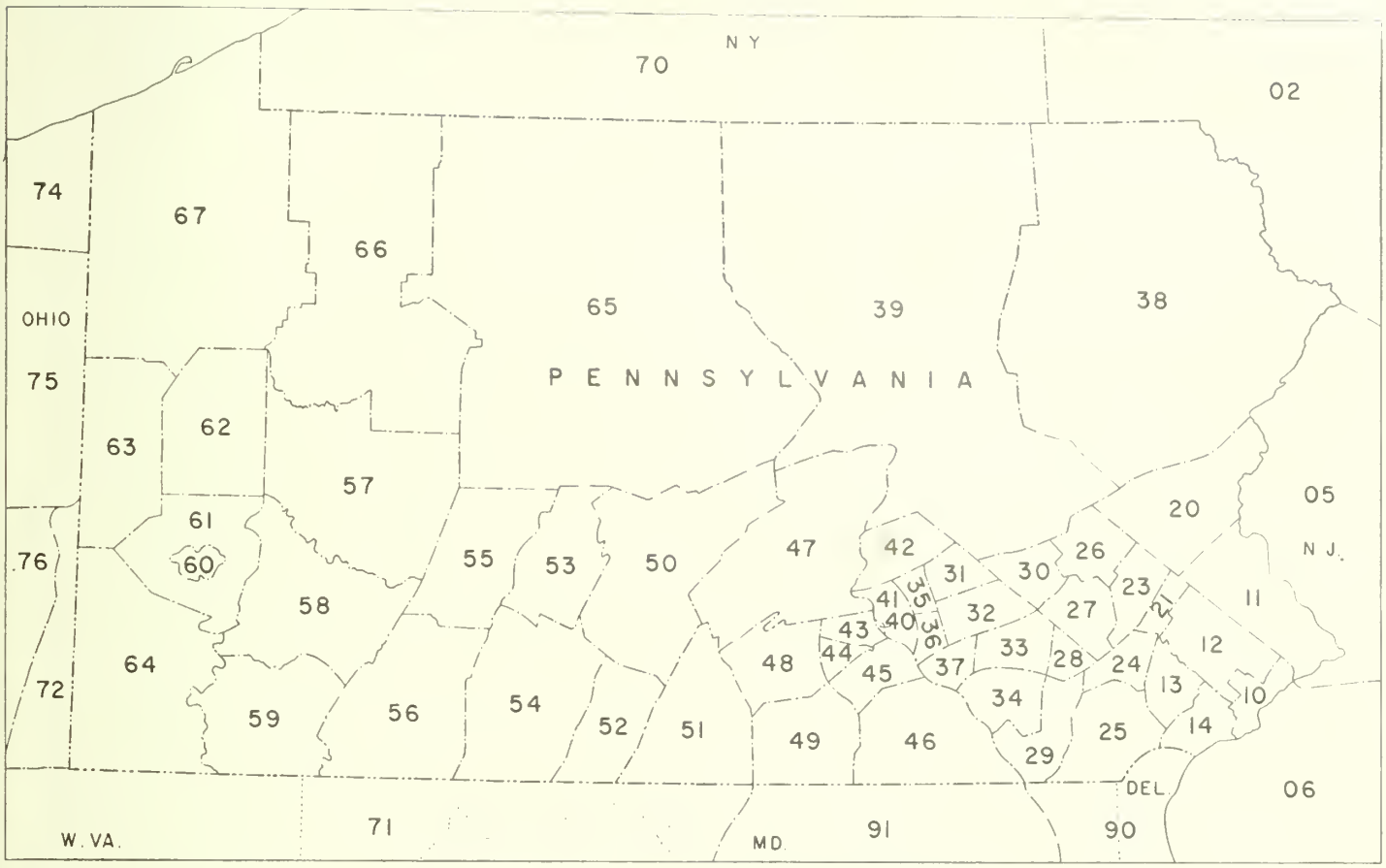


Figure 7.—Map of Pennsylvania indicating 2-digit zones.

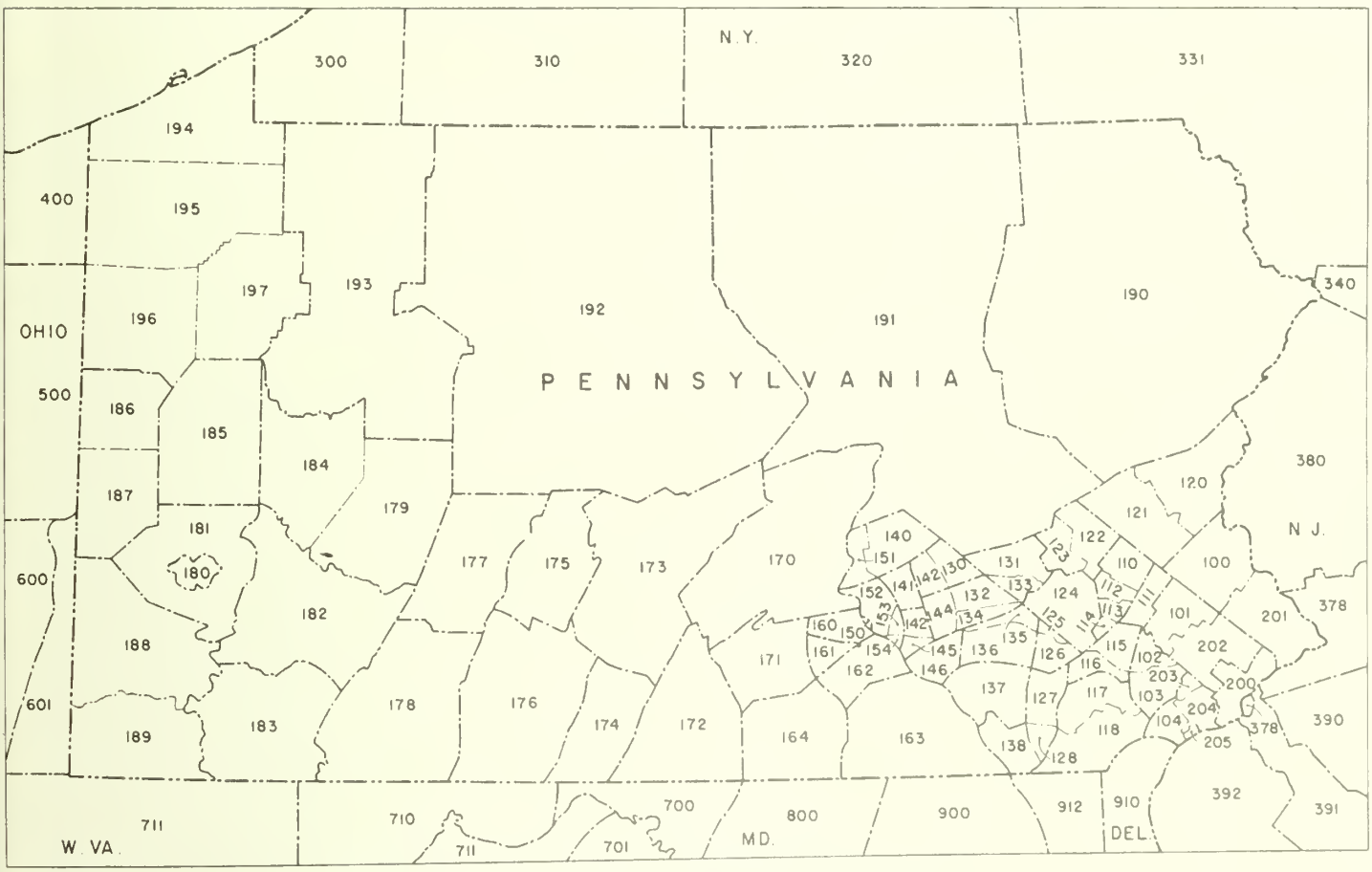


Figure 8.—Map of Pennsylvania indicating 3-digit zones.

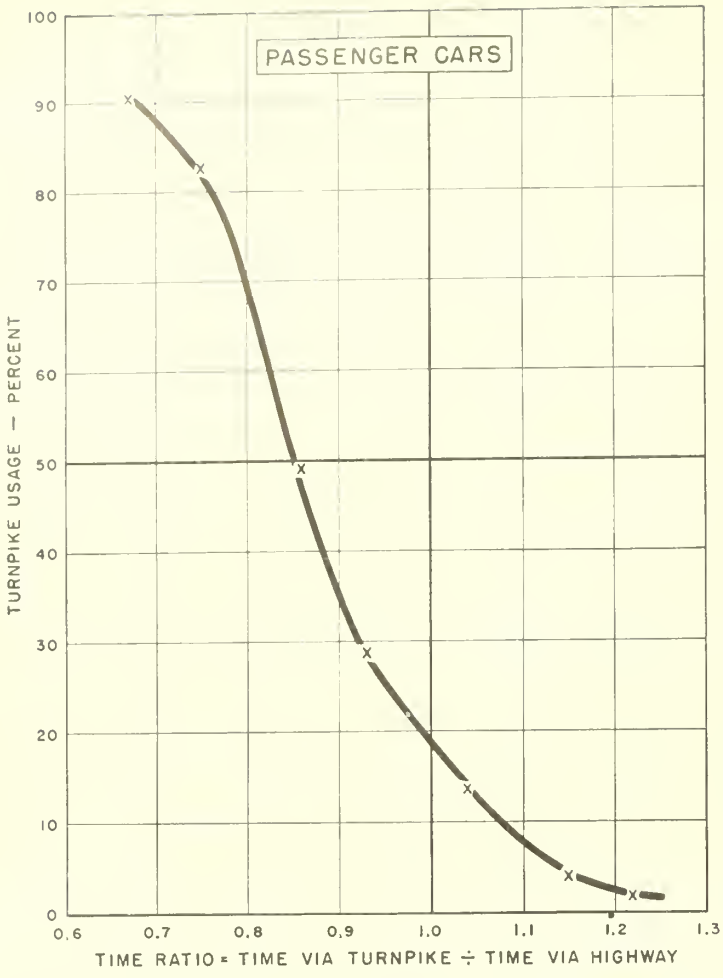


Figure 9.—Turnpike usage by passenger cars in relation to the time ratio.

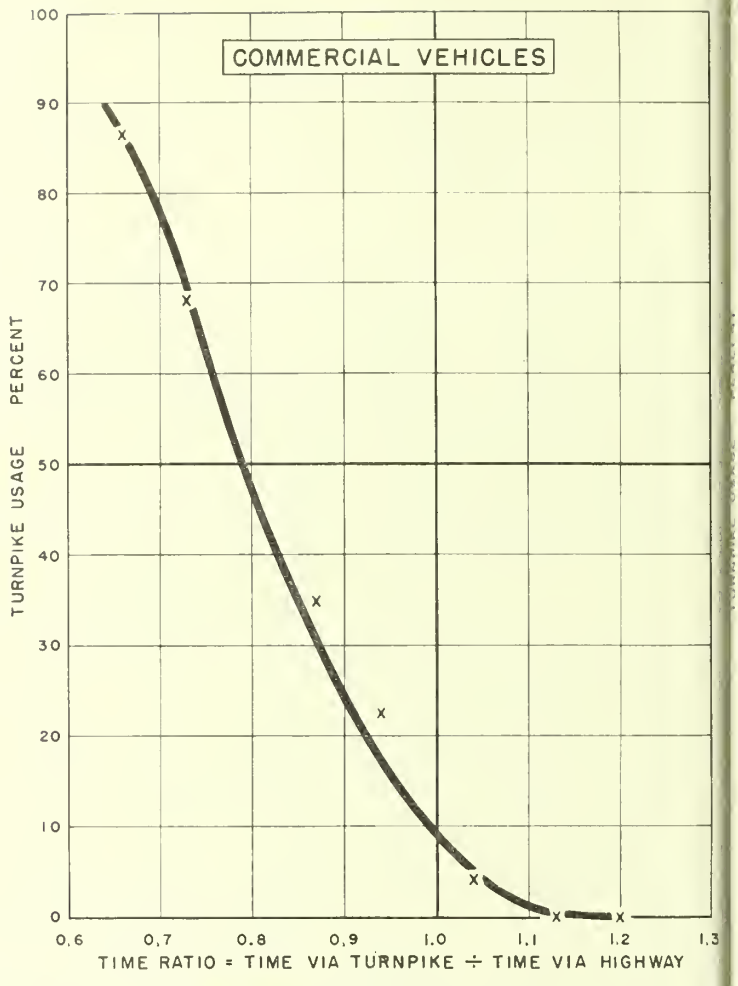


Figure 10.—Turnpike usage by commercial vehicles in relation to the time ratio.

at all, or were made by other means before the new facility was built. It is also possible that some trips were included which were diverted from roads so far away that they did not pass any of the survey stations. Some of the generated traffic occurs on free highways due to the transfer of former traffic to the toll facility, thereby making the old roads more attractive. The additional traffic, above diversion, that occurs on the new road, however, is of principal interest.

In addition to the diverted and generated traffic, it often happens over a period of years that additional traffic results from the development of new communities, industries, and businesses along the new highway. This development traffic, unless it occurs almost immediately after the opening of the facility, is not recognizable as generated traffic, but appears rather as a faster than normal growth.

Origin and Destination Zones

The comparison of travel time and distance on the old highways between major points or zones with the travel time and distance between the same points or zones by way of the new highway, and other relations based on these factors, provide tools for estimating the usage of a proposed facility. Trips between zones need to be analyzed to determine the percentages of trips that actually use the new and superior route.

As the basis for such an analysis, Pennsylvania as well as the rest of the country was divided into geographic zones. Comparatively small zones designated by three-digit numbers were first selected, but a preliminary review of the data indicated they were too small for proper analysis. After studying the detailed trip movements between individual zones, certain of the smaller zones were combined into two-digit zones to facilitate the analysis of the data.

Figure 6 shows both two- and three-digit zones for the United States except Pennsylvania. Two- and three-digit zones for Pennsylvania are shown in figures 7 and 8, respectively.

Time and Distance Data

Time and distance runs, "floating with traffic," were made on the turnpike and the highway routes between zones. The time and distance over logical free highways from origin to destination and via the turnpike, using the most logical free routes to and from the turnpike, were compared.

Based on nationwide speed studies and data from the Pennsylvania Turnpike Commission, the difference in speed between passenger cars and commercial vehicles was established as being generally 5 miles per hour. That is, if passenger cars traveled at 40 miles per hour, trucks were assumed to travel at 35 miles per hour. Where the speed was very low, 20 miles per hour or under, truck speeds were assumed to be the same as for passenger cars. The passenger-car speed on the turnpike was established at 57 miles per hour, and for commercial vehicles, 52 miles per hour on the eastern extension and 49 miles per hour on the remainder of the turnpike.

The average speed on the turnpike is approximately that of the driving speed. However, on regular highways the average speed is much below the driving speed on the open highway. It is for this reason that speeds used for free highways in this study are below speeds reported in general by State agencies and by the Bureau of Public Roads for rural highways. The speeds ordinarily reported are for level tangents on the open road, and do not reflect the time lost passing through towns and cities, or driving on tortuous curves and in hilly terrain. Time in this study includes that for passing through the many towns found in Pennsylvania, or from center to center of the municipalities.

An additional factor was involved for trips via the turnpike. Extra time and distance were required to get on the turnpike, pick up the ticket, pay the toll on leaving, and return to the regular highway. These actions added about 1 mile in distance and 4 minutes in time on the average. Consequently, these amounts were added to turnpike data before computing the time and distance ratios, and time or distance saving or loss.

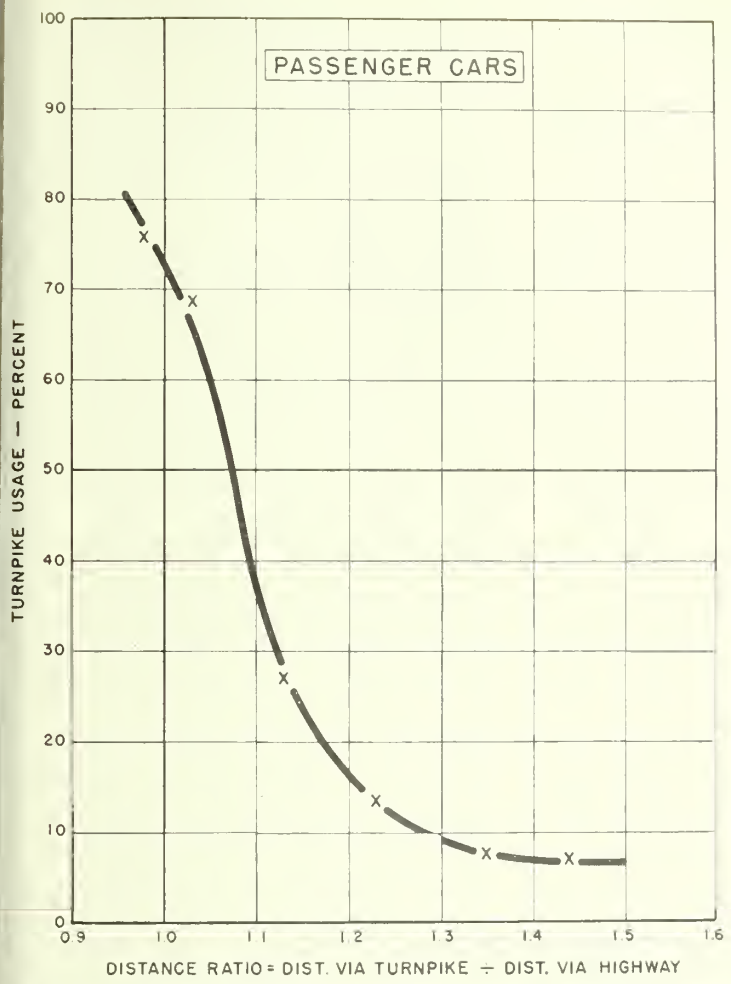


Figure 11.—Turnpike usage by passenger cars in relation to the distance ratio.

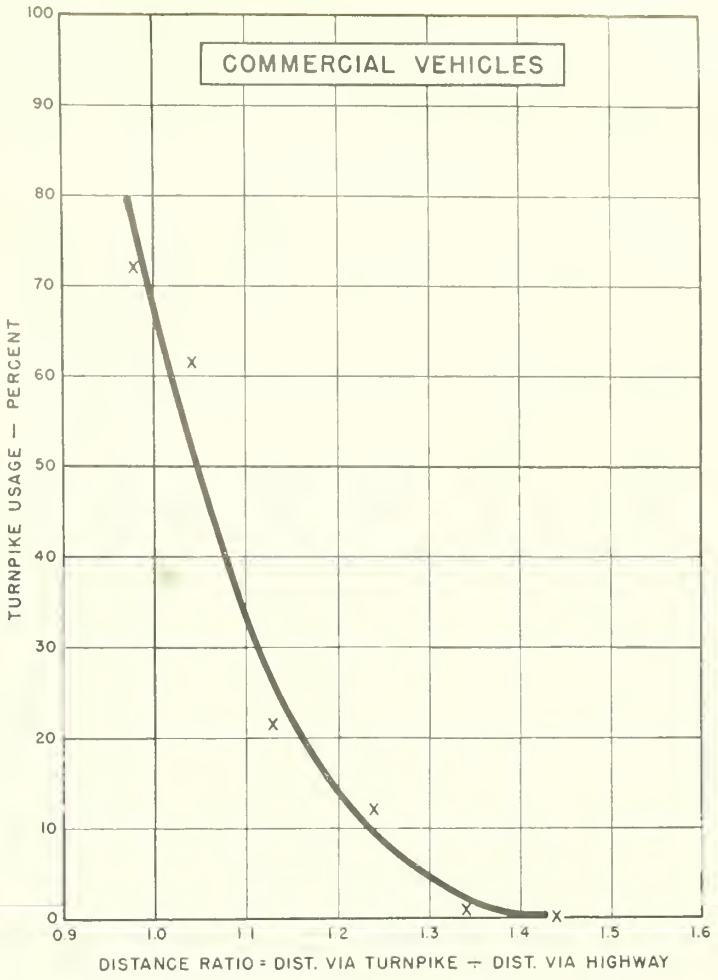


Figure 12.—Turnpike usage by commercial vehicles in relation to the distance ratio.

The data developed were on a zone-to-zone basis, as previously indicated, and the travel time and distance between the centers (usually, but not necessarily the geographic centers) of zones via logical routes and via the turnpike were computed. Where zones are large, it is possible that trips from that portion of the zone nearest the facility will use it to a larger extent than those farthest away. The use of actual origins and destinations for individual trips would result in more accurate ratios, but analysis in such detail would be time consuming, costly, and impracticable in this study. In any event, the application would ordinarily be by zones and the method used was, therefore, in keeping with the normal application of the results.

Experience in analysis of the Pennsylvania data indicates that the use of a toll road by drivers is not always determined on the basis of saving of time or distance. For example, it was found that some of the motorists paid toll to use the turnpike, even though they lost both time and distance in so doing. Some of them probably did not know whether they could save time by using the turnpike, while others perhaps did so intentionally preferring to use a high-type facility. Notwithstanding a certain amount of erratic behavior of this kind, time and distance relations do seem to be fairly reliable measures of the percentage use of the turnpike by trips between different pairs of zones.

Turnpike Usage Related to Time and Distance

The percentage of trips on the turnpike was determined in relation to time ratio (travel time via the turnpike divided by travel time via free routes), distance ratio (travel distance via turnpike divided by travel distance via free routes), products of these ratios, and amount of time saved and lost for passenger cars and commercial vehicles separately. So far as is known, this is the first time such relations have been established for a toll road.

First, the percentage of turnpike use was plotted separately for each zone-to-zone movement. This resulted in considerable scatter, largely because of the small number of trips between some of the zones. To determine the final curves for the various factors in relation to turnpike usage, it was therefore considered desirable to combine the ratios or minutes saved or lost into groups. These were averaged both for turnpike usage and for other factors to obtain central points through which the final curves were drawn.

In most instances, it was found that a simple average of the percentage turnpike use between pairs of zones falling into the same group of ratios or minutes was preferable to an average weighted in accordance with the number of trips between each pair of zones. This was likely due to the very large number of trips from certain urban zones where the usage ratio was not comparable to those for other zones. By weighting in such cases, the unusual movement had entirely too much influence on the curve.

Turnpike Usage Related to Time Ratio

Time ratios for trips between the various zones were computed for passenger cars and commercial vehicles separately. The percentage of turnpike use was plotted against the time ratios, grouped as previously explained. The resulting curves are shown in figures 9 and 10.

The time ratio was computed by dividing the travel time for the toll facility by the travel time for regular highways. The time required to travel the separate routes was obtained from runs made by the floating-car method as previously indicated.

In case the trip extended beyond the ends of the turnpike, time and distance were not figured for the entire trip in computing ratios, but for only that portion within the turnpike area. In this procedure, "points of choice" were designated; that is, for computation purposes, the termini of the longer trips were considered as the points where drivers made a choice of whether the turnpike would be used. The reason for using this procedure is that travel time and distance for the portion of a trip lying beyond the turnpike area are the

same for trips made on the turnpike and those which are not, and the ratios tend to approach unity as distance beyond the turnpike area increases.

The effect on time ratios of that portion of the trip not eligible to use the turnpike may be illustrated as follows: Passenger-car trips between the Ohio State line and Philadelphia have a time ratio of 0.65 for which the curve in figure 9 indicates a 92-percent turnpike usage. An extra 100 miles beyond the turnpike causes an increase to 0.73 of the time ratio with a consequent reduction of indicated turnpike usage to 85 percent as determined from the curve. An increase in trip length of 1,000 miles increases the time ratio to 0.91 and reduces the indicated turnpike usage to 33 percent.

This means that the three groups of trips considered, which could be made on the turnpike between the Ohio line and Philadelphia and save the same amount of time, would have three different time ratios, and the turnpike usage on the basis of the time-ratio curve would be 92, 85, and 33 percent, respectively.

It has been definitely established that approximately the same percentage of trips are made on the Pennsylvania Turnpike between zones west of the Ohio line and Philadelphia or east of the Delaware River, regardless of whether the origin is near the turnpike or at points as far away as the west coast or New England. Therefore, for trips with termini beyond the turnpike where drivers could logically use either the turnpike or a parallel road, a point of choice was established before computing time and distance ratios. Trips with origin or destination west of Pennsylvania were assigned the Ohio line as an approximate point of choice. Deerfield, Ohio, was used for points to the north, such as Cleveland, Ohio, and Michigan; Canton, Ohio, for trips normally using U. S. 30; and Washington, Pa., for trips between the southwest and east that pass through the Pennsylvania corridor. It was found that a point of choice appeared to be near Somerville, N. J., for trips with origin or destination in northern New Jersey and beyond Somerville.

Because the points of choice are definite points on the highway, the time and distance ratios for trips extending beyond the turnpike area are more accurate than the ratios for trips with both termini within the turnpike area, where time and distance are measured from center to center of zone. In the latter case, the time and distance for many of the trips differ materially from the measurements because origins and destinations may be near the edge rather than the center of the zone. This is one of the factors causing the previously mentioned scatter in the plotted points for zone-to-zone movements.

There are other important factors, however, contributing to this scatter. Drivers do not all travel at the same speed, so the time ratios between the same points vary among drivers. Also, the ease of driving on the turnpike appeals to some more than others and where a toll is involved, the financial status of individuals has an important bearing on the choice of route.

Because of these factors, it is impossible to determine whether a particular trip will be made on the turnpike, particularly if the time ratio is between 0.85 and 1.05, but it is possible to determine with a reasonable degree of accuracy the percentage of drivers that will do so. Within reasonable limits, therefore, it is possible to assign trips to a turnpike on the basis of the time ratio.

In the method of grouping used, the variations just mentioned have been ironed out and the plotted points for the groups shown as X's in the figures fall very close to a smooth curve, as in figures 9 and 10, for example. Before deciding upon the groupings shown, other smaller groupings were tried and, in some instances, the shape of the curve was influenced to some extent by these smaller groupings.

From the curve in figure 9, the proportion of turnpike usage can be read for the time ratios computed for travel of passenger cars between zones. For example, when the time ratio was 0.67 or less, 90 percent or more of the trips were made on the turnpike. For a time ratio of 0.80, the percentage use approximated 69 percent; for 0.90, it was 35 percent; for 1.00 or equal time via the turnpike and via the free road, it approximated 19 percent. Because of higher speeds the distance via the turnpike was, of course, greater than the distance via the free routes when travel time was equal; and it would appear that almost one-fifth of the drivers traveled this extra distance, saved no time, and yet paid toll. This is not necessarily true, however, because some of these drivers may actually have had lower time ratios because of the location of their origins and destinations within zones, driving habits, or other factors. The turnpike usage dropped to 7 percent at time ratio 1.10 and was down to about 2 percent at 1.20.

The curve for commercial vehicles shown in figure 10 was drawn on the basis of groupings similar to those used in plotting figure 9 for passenger cars. The curve for commercial vehicles is somewhat steeper, and there is, on the whole, a lesser percentage of turnpike use for commercial vehicles than for passenger cars. For the whole curve, assuming even distribution throughout, approximately three-fourths as great a percentage of commercial vehicles as passenger cars used the turnpike. This percentage, of course, varies according to where the trip-time ratio falls on the curve.

Figure 14 (Right).—Turnpike usage by commercial vehicles in relation to the product of time and distance ratios.

Table 3.—Standard error of estimate of percentage of turnpike use based on the variation of points from a final curve in each category

Turnpike usage criteria	Standard error of percentage of turnpike use for—	
	Passenger cars	Commercial vehicles
	Percent	*Percent*
Time ratio:		
Individual points	12.3	17.1
Averages of—		
0.01 ratios	6.2	11.9
0.05 ratio groupings	2.2	4.3
0.10 ratio groupings	.9	2.8
Distance ratio:		
Individual points	21.2	24.2
Averages of—		
0.01 ratios	11.8	11.6
0.05 ratio groupings	4.4	5.3
0.10 ratio groupings	1.2	4.3
Product of time and distance ratios:		
Individual points	13.9	17.8
Averages of—		
0.01 ratios	10.3	13.2
0.05 ratio groupings	4.3	3.8
0.10 ratio groupings	2.7	1.3
Product of time ratio squared and distance ratio:		
Individual points	12.6	18.2
Averages of—		
0.01 ratios	13.3	16.6
0.05 ratio groupings	4.1	6.5
0.10 ratio groupings	2.7	2.9
Time differential (time saved or lost):		
Individual points	11.9	13.5
Averages of groupings by—		
5-minute intervals	8.5	12.9
10-minute intervals	7.8	11.7
15-minute intervals	3.4	11.1
30-minute intervals		5.6

Table 4.—Through trips of passenger car, commercial, and total vehicles between selected cities, together with the number and percentage of trips using the turnpike on an average summer weekday, 1950–52

Year and location	Passenger cars			Commercial vehicles									All vehicles		
	Using turnpike		Total	Combinations			Other commercial			All commercial			Using turnpike		Total
	Number	Percent of total		Using turnpike		Total	Using turnpike		Total	Using turnpike		Total	Number	Percent of total	
				Number	Percent of total		Number	Percent of total		Number	Percent of total				
1950:															
Pittsburgh–Philadelphia	171.1	87.3	196.1	170.7	90.7	188.1	33.7	87.3	38.6	204.4	90.2	226.7	375.5	88.8	422.8
1951:															
Pittsburgh–Philadelphia	209.4	91.0	230.1	171.8	95.6	179.7	36.7	80.5	45.6	208.5	92.5	225.3	417.9	91.8	455.4
Pittsburgh–Harrisburg	184.9	89.7	206.1	37.3	94.2	39.6	6.5	100.0	6.5	43.8	95.0	46.1	228.7	90.7	252.2
1952:															
Pittsburgh–Philadelphia	227.1	88.5	256.5	162.6	86.0	189.1	26.9	82.8	32.5	189.5	85.5	221.6	416.6	87.1	478.1
Harrisburg–Philadelphia	497.6	75.6	658.2	25.4	22.6	112.3	37.9	59.4	63.8	63.3	35.9	176.1	560.9	67.2	834.3

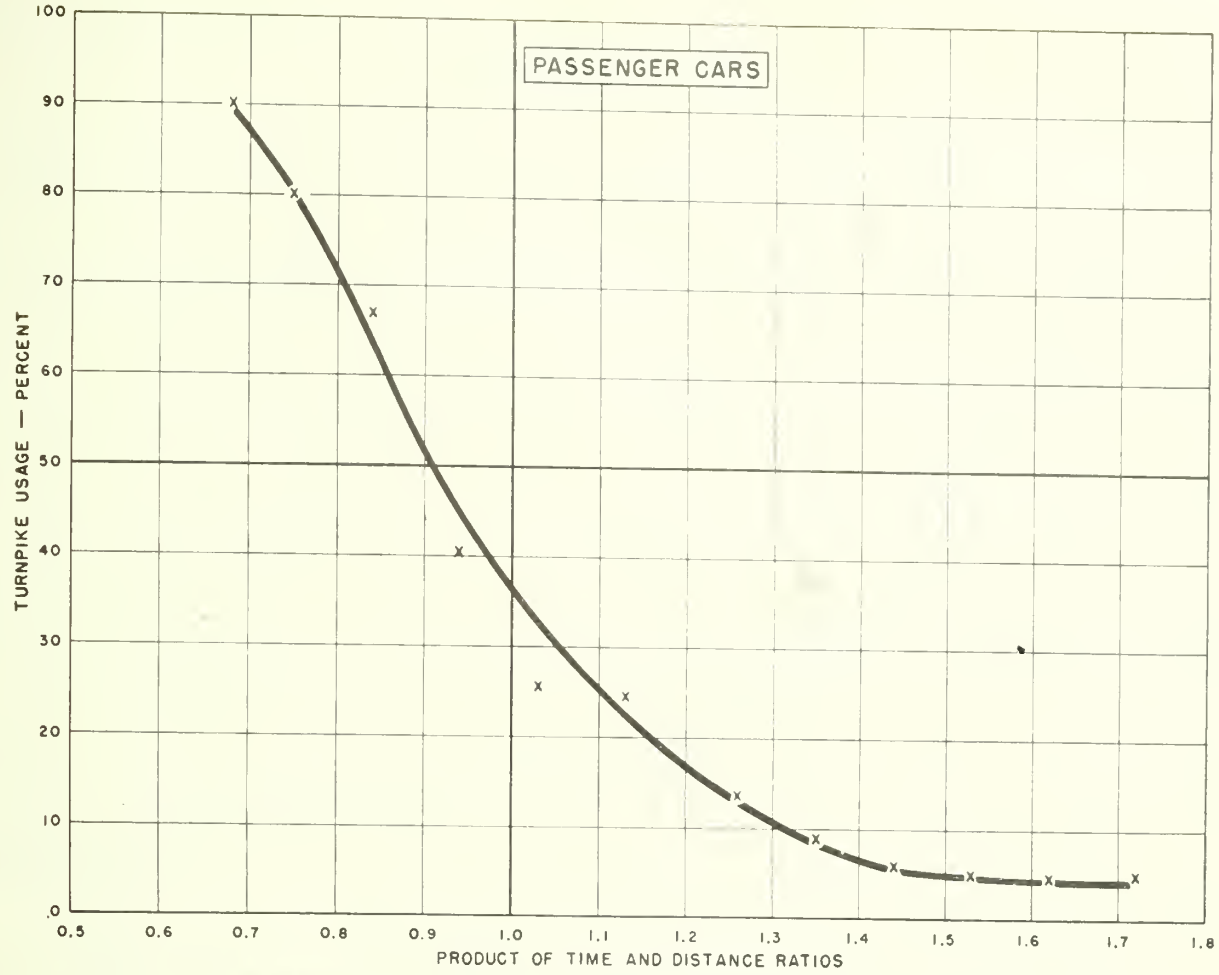
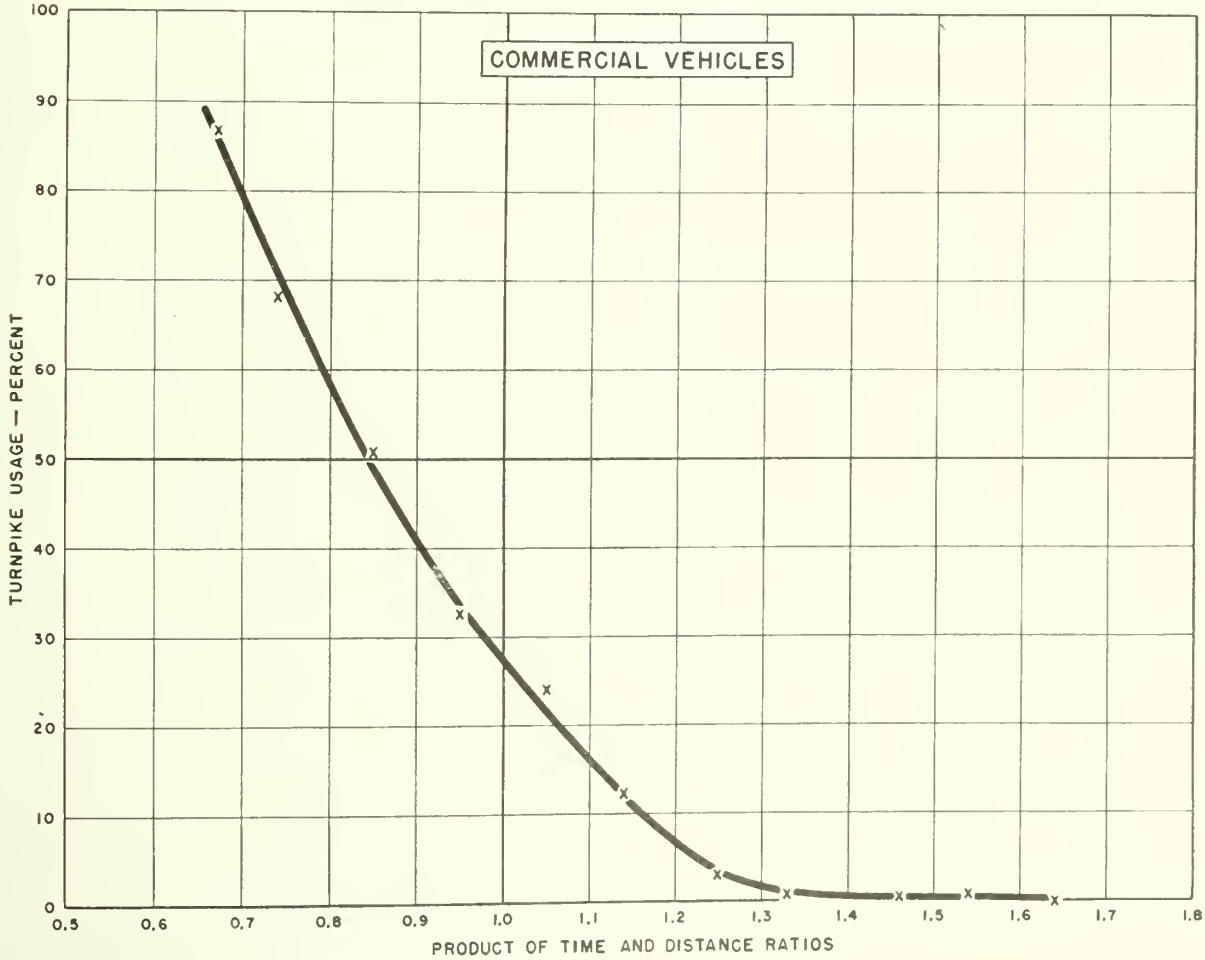


Figure 13.—Turnpike usage by passenger cars in relation to the product of time and distance ratios.



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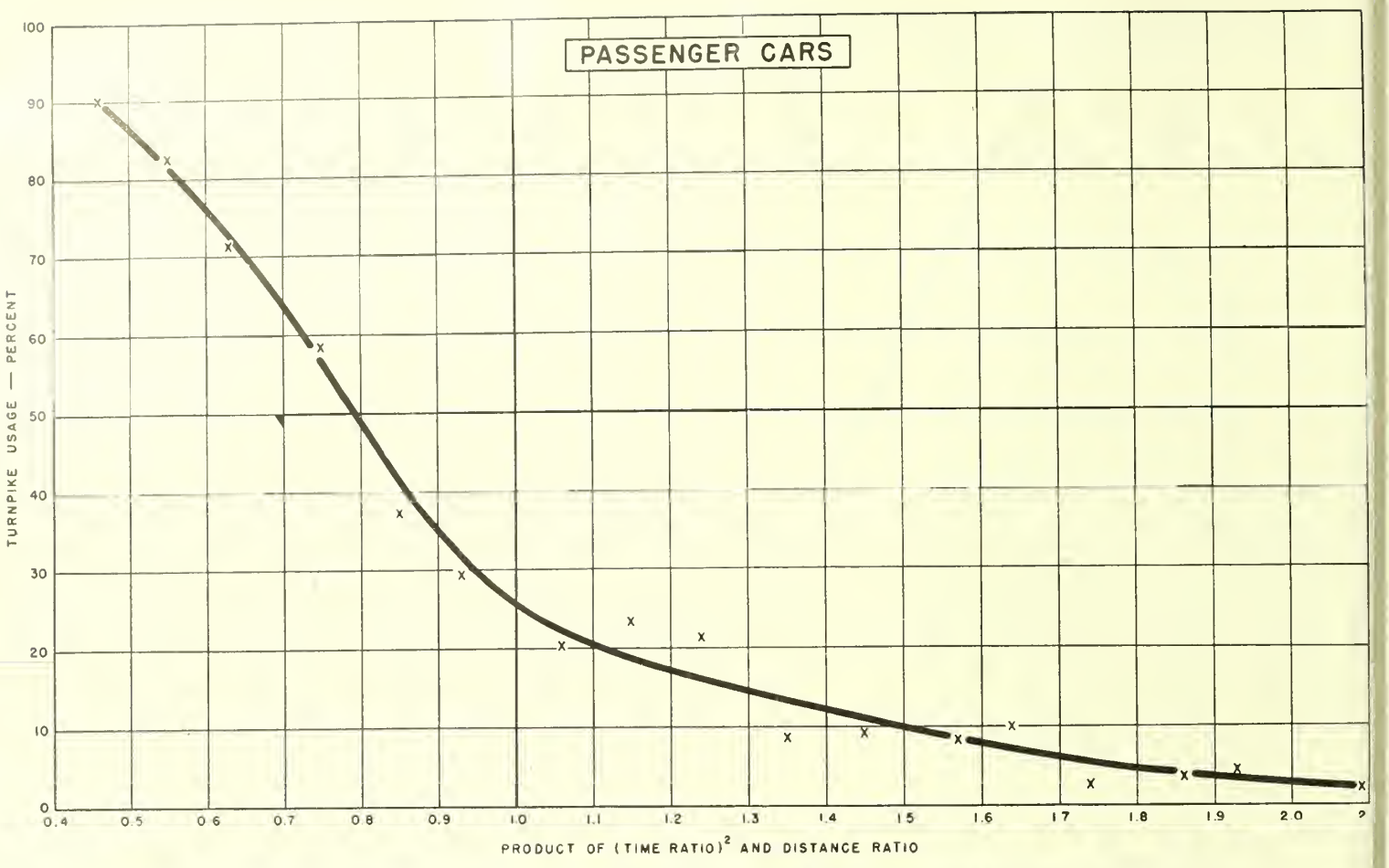

Figure 15.—Turnpike usage by passenger cars in relation to the product of time ratio squared and distance ratio.

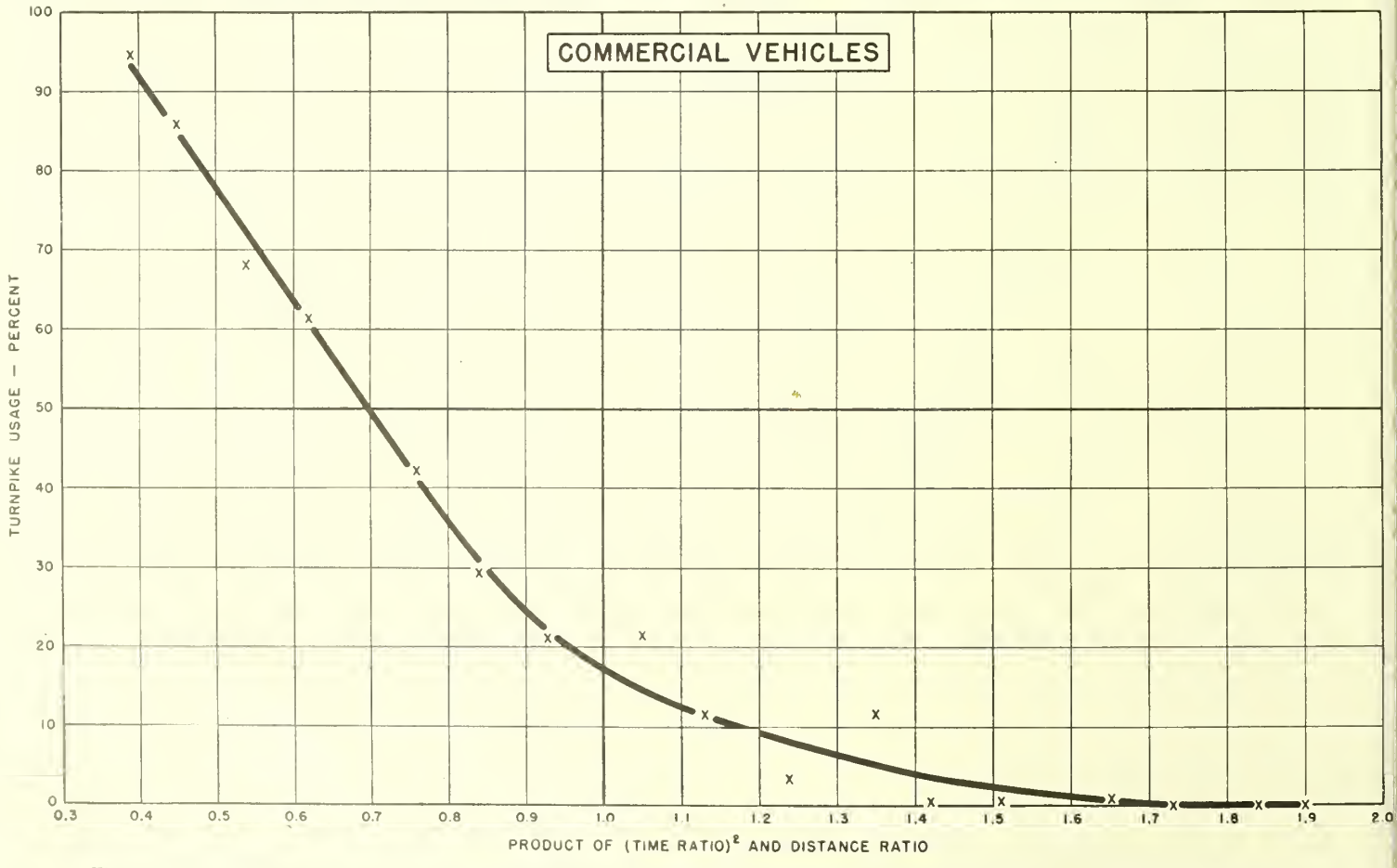

Figure 16.—Turnpike usage by commercial vehicles in relation to the product of time ratio squared and distance ratio.

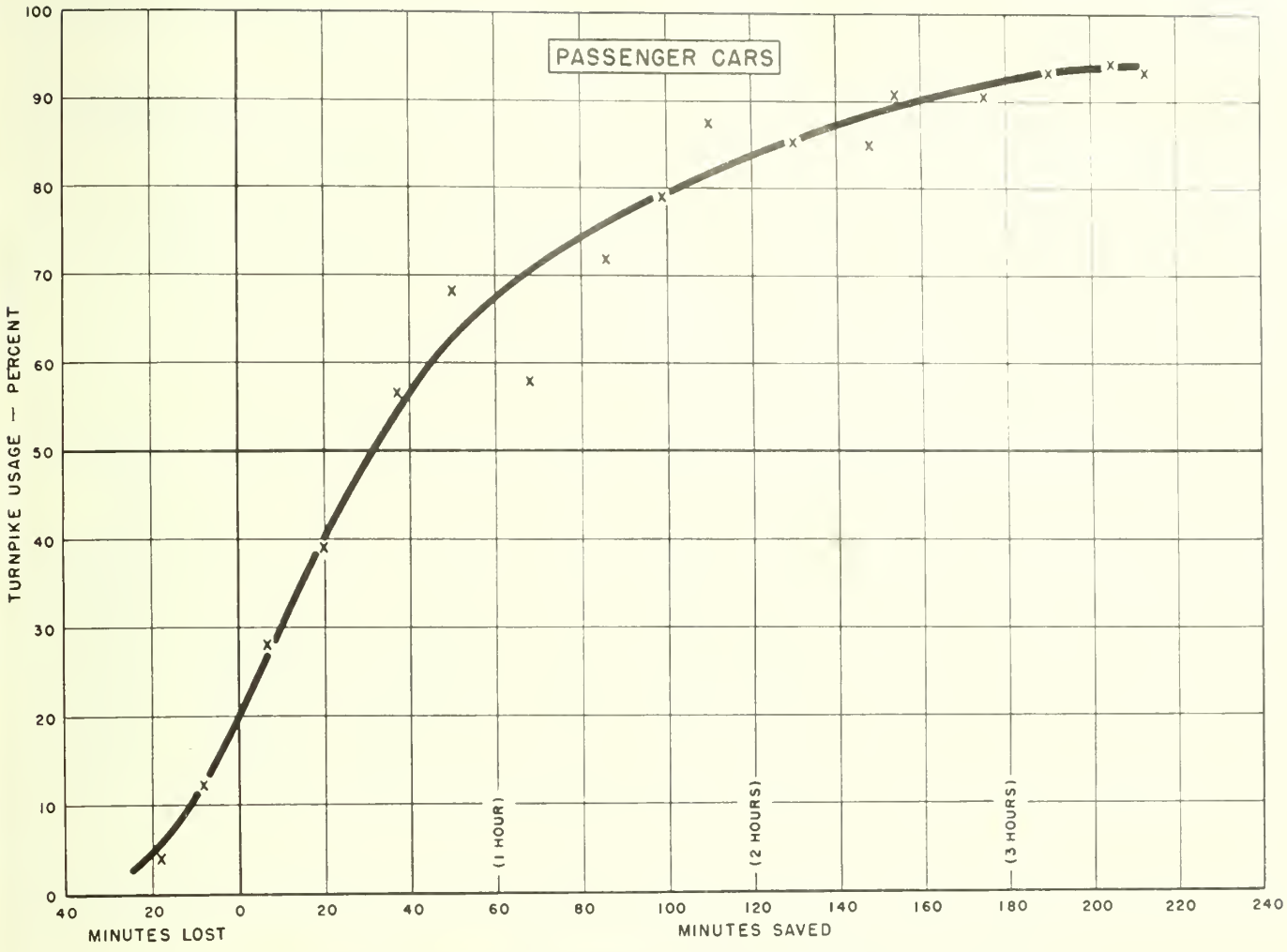

Figure 17.—Turnpike usage by passenger cars in relation to the time differential.

The percentage use for commercial vehicles is much less than that for passenger cars as the time ratio increases to 0.80 and above, with only one-third as much as for passenger cars at a ratio of 1.05.

A comparison of turnpike usage with that of a free road of similar design is valuable because it may give some indication of the effect of tolls on the usage of such a facility. A curve showing the percentage use of a portion of the Shirley Memorial Highway, a free fully controlled-access road in Virginia entering Washington, D. C., was developed in a previous study.[5] If the curve developed for the Shirley Highway had been applicable to the Pennsylvania Turnpike, 62 percent of the eligible trips would have been made on the facility, whereas only 44 percent actually did so. In other words, approximately 70 percent of the trips that would have been made on such a free road had the Shirley Highway curve been applicable, actually were intercepted on the turnpike and paid toll. We cannot conclude that this percentage is an accurate measure of the effect of the toll charge on turnpike usage, however, as there are other important differences between the two highways. The Shirley Highway is shorter and has more frequent interchanges than the Pennsylvania Turnpike. Even more important, perhaps, is the fact that it is located largely in an urban area, whereas the area through which the turnpike passes is dominantly rural.

[5] *The effect of travel time and distance on freeway usage*, by Darel L. Trueblood. PUBLIC ROADS, vol. 26, No. 12, Feb. 1952.

Turnpike Usage Related to Distance Ratio

Distance ratios were computed in a manner similar to that for time ratios already described except that distances were substituted for time. For trips extending beyond the turnpike limits, distance ratios were computed from point of choice, as in the case of time ratios. Turnpike usage in relation to distance ratios for passenger cars is shown in figure 11.

Few of the distance ratios in Pennsylvania are less than 1.00. Where the distance is the same, a very large percentage of vehicles will be found on the turnpike, since in such case there is a substantial saving in time. In fact, at a distance ratio of 1.00 and at a time ratio of 0.79, about 73 percent of the passenger cars in Pennsylvania, on the average, used the turnpike. As in the case of the time-ratio chart (fig. 9), the distance-ratio chart has critical areas where the scatter is rather large when plotted for individual zone-to-zone trip interchanges, but by properly combining the data a logical series of points can be obtained to establish a curve.

The comments concerning distance ratios for passenger cars apply in general to commercial vehicles. However, for commercial vehicles, the shape of the distance-ratio curve (fig. 12) is more nearly like the time-ratio curve (fig. 10) than was the case for passenger cars. Of course, the values for distance ratio are greater. Where the distances are equal via the turnpike and via highways; i. e., ratio 1.00, about 68 percent of the total commercial vehicles used the turnpike. This is 5 points less than at the same distance ratio for passenger cars. There was a considerably lower percentage of commercial vehicles using the turnpike than in the case of passenger cars when the distance ratio was 1.25 or greater.

Turnpike Usage Related to Product of Time and Distance Ratios

In figure 13 is presented a curve indicating the turnpike use for the several values of the product of the time and distance ratios for passenger cars. This curve is less steep than either the time- or distance-ratio curves for passenger cars. The ratio ranges from less than 0.70 to more than 1.70 or almost twice the spread for the time ratio.

The turnpike usage as read from this curve amounts to about 90 percent at 0.68, which is practically the same as indicated by the time-ratio curve. However, at ratio 1.00, 36 percent instead of 19 percent used the turnpike. There was a 4-percent usage of the turnpike at product ratio 1.70, while for the time ratio this percent usage occurs at 1.16.

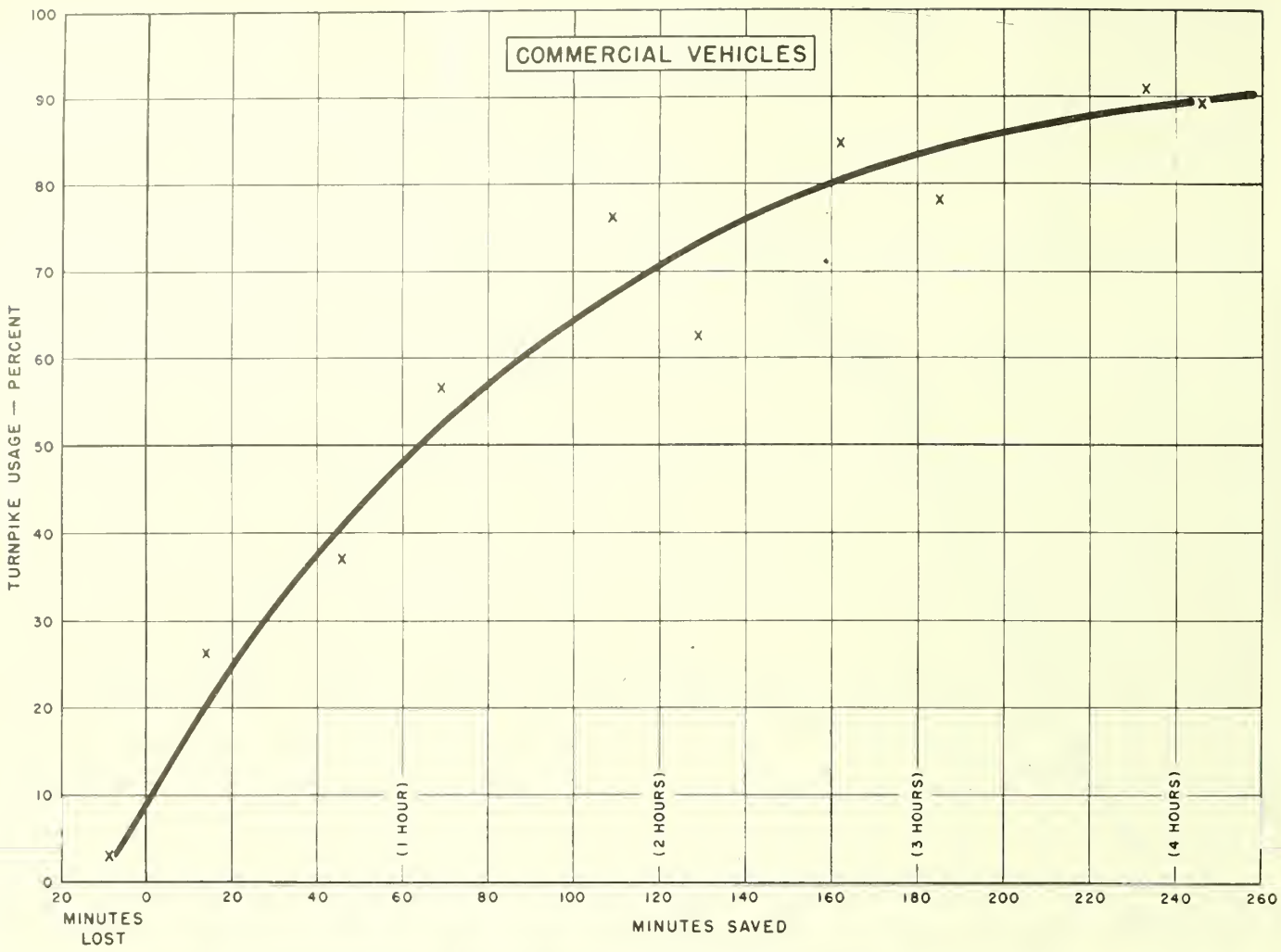

Figure 18.—Turnpike usage by commercial vehicles in relation to the time differential.

The curve for turnpike usage for commercial vehicles is shown in figure 14. This is somewhat similar to the passenger-car curve, but a little steeper. Also, the turnpike usage is almost nil after the ratio reaches 1.35, whereas the passenger-car curve still shows 4-percent usage at 1.70.

The difference in turnpike usage between the two curves is about 13 percentage points at 0.80 and 4 points at 1.60 and above. At 1.00 ratio 27 percent of the commercial vehicles used the turnpike as compared with 36 percent for passenger cars. At 1.00 the difference is, therefore, 9 percentage points, but this is equivalent to 33 percent more passenger cars than commercial vehicles.

Usage Related to Product of Time Ratio Squared and Distance Ratio

The product of time and distance ratios has a tendency to flatten the curve in comparison with either time-ratio or distance-ratio curves. Time ratio may be more indicative of turnpike usage than distance ratio, since the distance ratio between two zones remains constant while the time ratio may vary for the same zones from time to time because of changes in speed. For example, the time ratio may be reduced with consequent increase in turnpike use when the parallel free road becomes congested to the extent that driving time is increased thereon.

This being true, it was thought that assigning more weight to the time ratio might improve the curve based on the product of time and distance ratios. After much experimenting, it was found that the square of the time ratio multiplied by the distance ratio ($T^2 \times D$ ratios) gave good results. Such a curve is shown in figure 15 for passenger cars. The range in ratios is from less than 0.50 to over 2.00, or almost 3 times the range for time ratio (fig. 9).

The curve for commercial vehicles, figure 16, is quite similar to the one for passenger cars but, as usual, the commercial usage of the turnpike is less than for passenger cars for the same ratio. The commercial-vehicle curve ranges from less than 0.40 to 1.90. Actually the turnpike usage becomes practically zero at 1.70 and above. The passenger-car curve shows 2 percent use of the turnpike at 2.10.

At ratio 1.00, more than 25 percent of the passenger cars use the turnpike, while only 17 percent of the commercial vehicles do so for the same value.

Usage Related to Time Differential

The curve showing turnpike usage on the basis of time saved or lost for passenger cars is shown in figure 17. Like all of the usage curves, there was considerable scatter when individual zone-to-zone movements were used. By grouping into 15-minute intervals the data became sufficiently well stabilized so that a smooth curve could be drawn. With smaller time groupings, a number of points were considerably out of line, but for portions of the curve where this was not the case they were sometimes used to aid in determining the exact shape of the curve.

It will be noted that some trips were made on the turnpike even with a 20-minute loss in time, which means a considerable loss in distance. With a saving of 20 minutes, approximately 40 percent of the drivers paid toll in preference to using the regular highways; this percentage increases to over two-thirds of the trips with an hour's saving in time, and to 93 percent with a 3-hour saving.

The minutes-saved curve for commercial vehicles is shown in figure 18. The scatter pattern for trucks is such that the groupings in number of minutes saved is double that for passenger cars (30 minutes instead of 15) before stability results.

For commercial vehicles the use of the turnpike was less than for the same time saving for passenger cars. For 20-minute time saving there was 25 percent usage for trucks compared with 40 percent for passenger

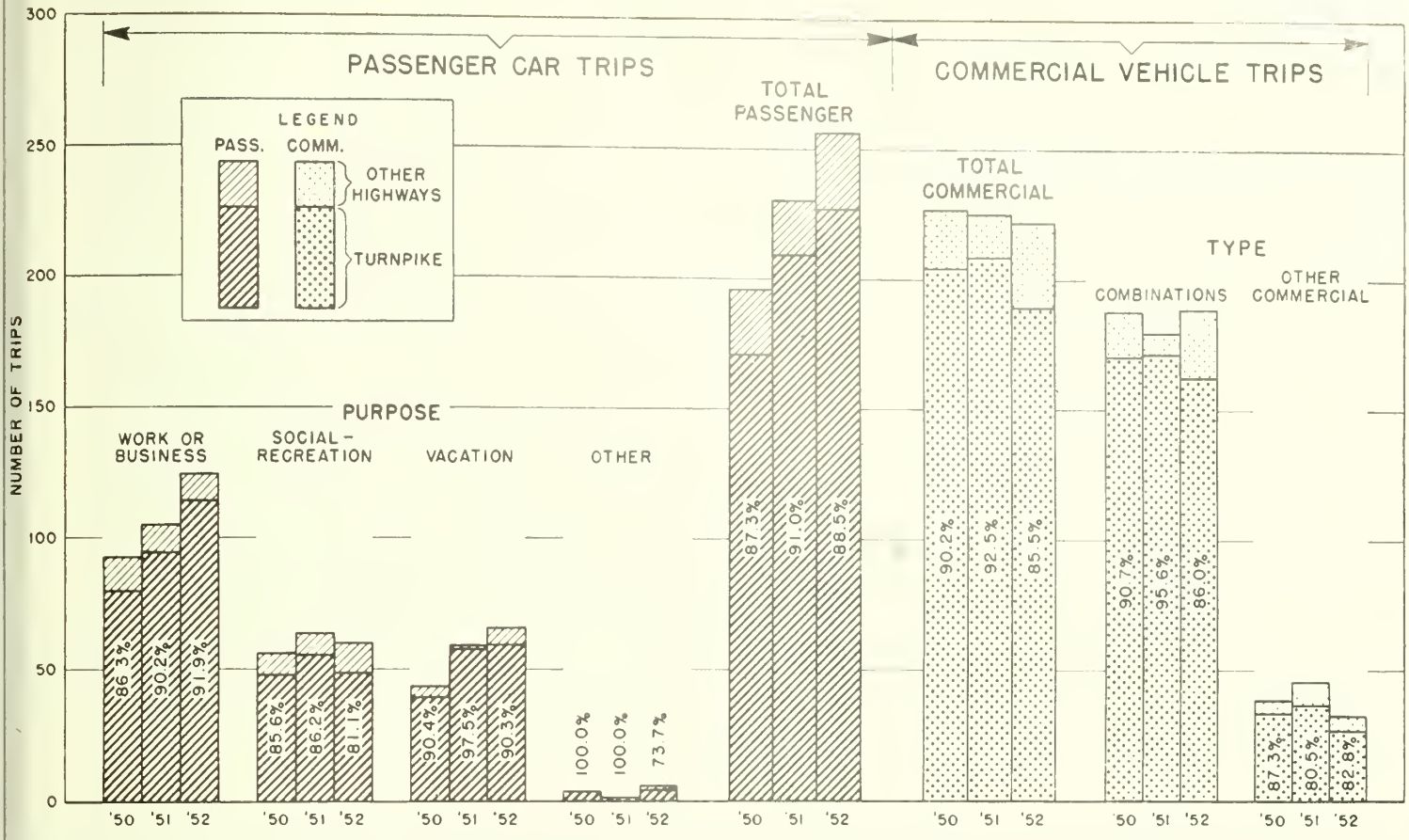

Figure 19.—Number of through trips between Pittsburgh and Philadelphia for passenger cars and commercial vehicles on an average summer weekday, related to turnpike usage and trip purpose (passenger cars) for years 1950–52.

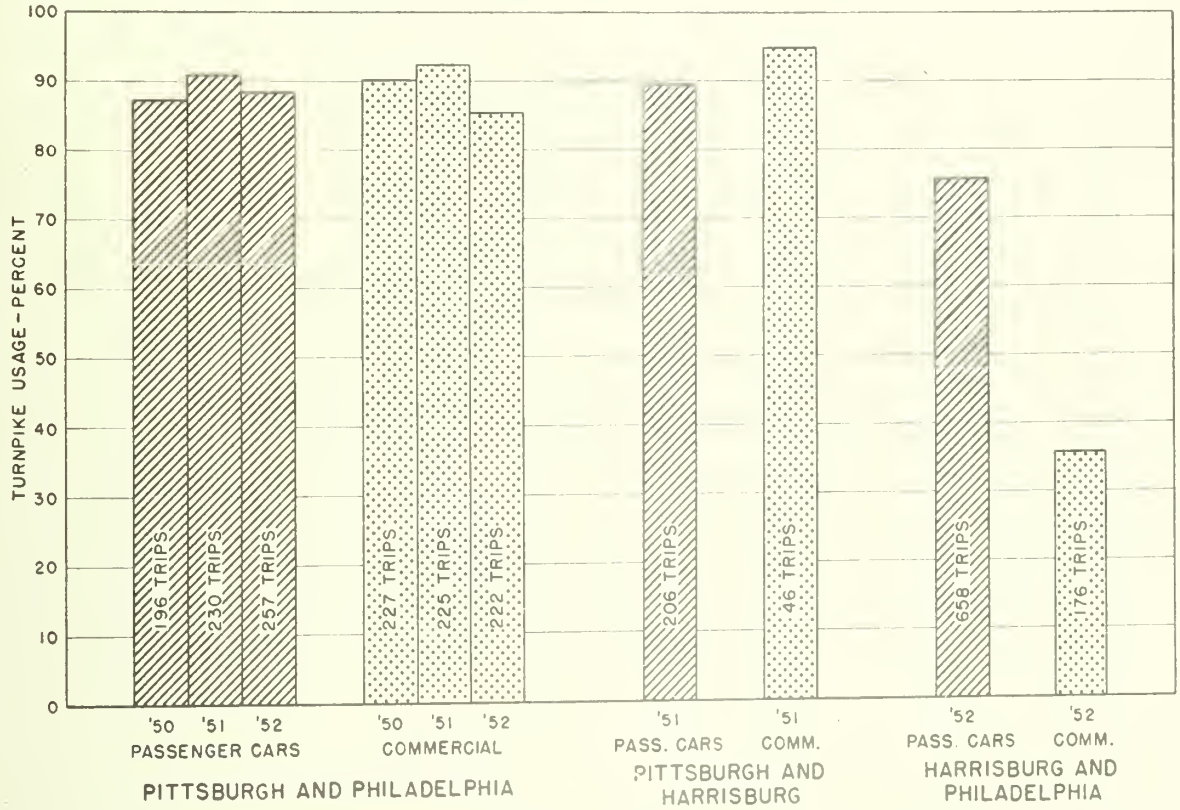

Figure 20.—Turnpike usage for through trips between selected cities on an average summer weekday.

cars; for 1-hour time saving, a 48 percent usage compared with 67 percent for passenger cars; for a 2-hour time saving, a 71 percent usage compared with 83 percent for passenger cars; and for a 3-hour saving, an 83 percent usage compared with 93 percent for passenger cars. No trucks losing 10 minutes or more used the turnpike.

Standard Error of Estimate

The standard error of estimate has been computed for each of the curves in this report

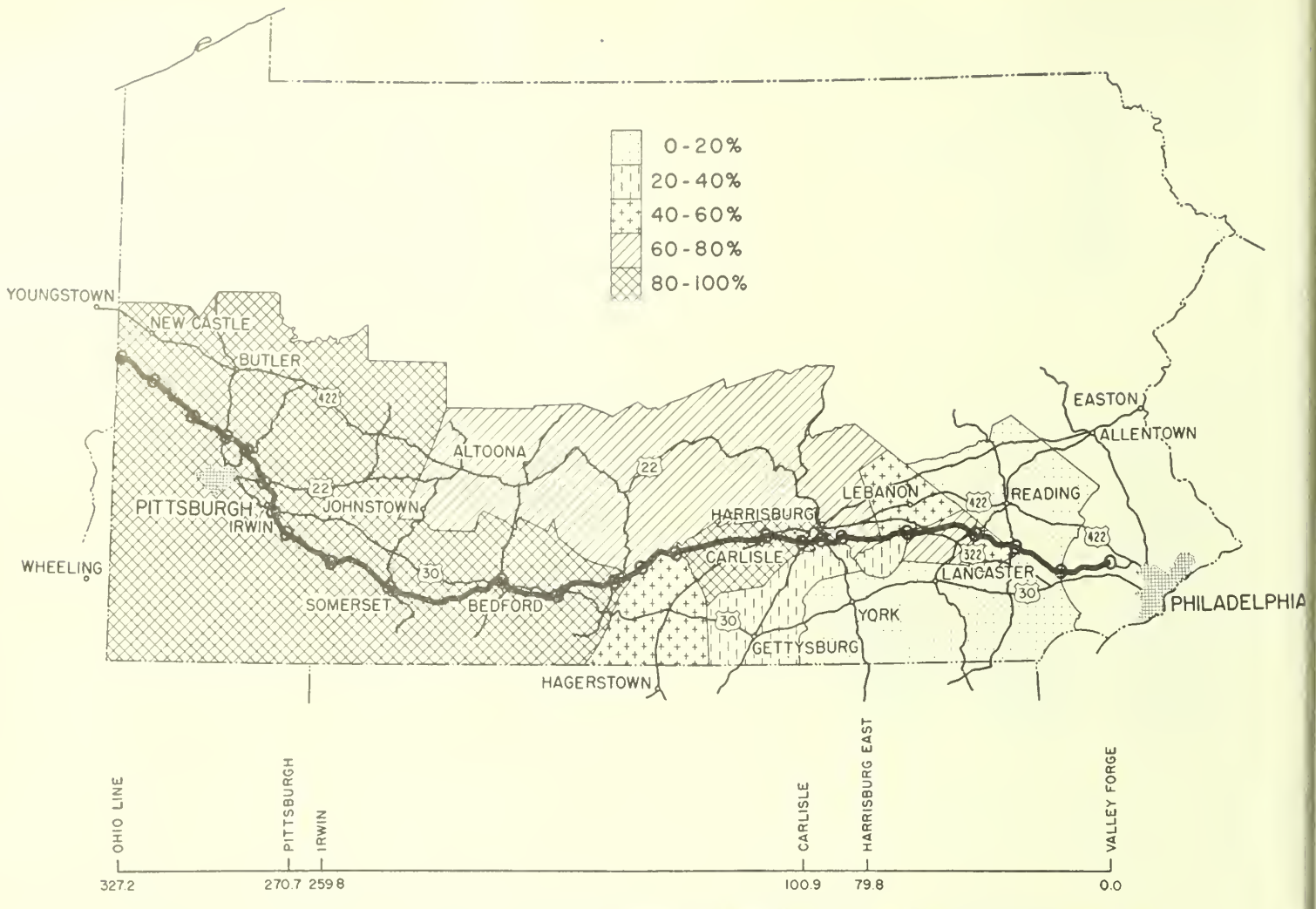

Figure 21.—Percentage of passenger cars using the turnpike between Philadelphia and areas along the turnpike route on an average summer weekday in 1952.

as a measure of the scatter of the averages of the grouped data from the fitted curve. Curves plotted parallel to a fitted curve and at a distance of plus or minus 1 standard error from it will contain within their bounds approximately two-thirds of the points to which the curve was fitted.

In general, original observations have a greater degree of scatter than averages for groupings of the observations. Computing group averages prior to curve fitting has the effect of smoothing the data. This is justifiable if thereby the functional relation is made apparent. However, care must be taken lest excessive smoothing obscure the effects of factors which should be taken into account.

In actual practice the origin and destination data are grouped in zone-to-zone movements. Unfortunately the trip interchange between zones is not uniform, varying from only a few trips to several thousand trips. The initial standard error was computed on the basis of the individual zone-to-zone movements notwithstanding the fact that some of the zone-to-zone trips were so few that they hardly constituted a sample of adequate size.

To mitigate the effect of the small sample, the data were then successively grouped into large units. In the case of ratio abscissa the trips within each 0.01 ratio were added together to give one point, then within each 0.05 ratio to give one point, and finally within each 0.10 ratio to give one point. In the case of time differential, similar groupings were successively made within 5-minute, 10-minute, 15-minute, and 30-minute intervals. The results of these calculations for both passenger cars and commercial vehicles are tabulated in table 3.

For the individual zone-to-zone movements, time differential has slightly greater accuracy than the other curves for both passenger cars and trucks. However, as the data are grouped the time-ratio curve appears to give the best results. In addition, time differential is inherently dependent on the length of the turnpike and would, therefore, require a series of curves for each length of turnpike. Time ratio, on the other hand, is independent of length and is more generally applicable, particularly in view of the small difference in standard error.

The computation of standard error is of importance mainly in the comparison of methods. In the application of the results, the interest lies in the accuracy of the total volume which will use a facility rather than in the accuracy of the individual zone-to-zone movements.

Since any estimate of total turnpike volume would involve a large number of zone-to-zone movements, the proportion the standard error of the total bears to the total volume would be substantially smaller than the proportion the standard errors of estimates of zone-to-zone movements bear to zone totals. In absolute amount the former would be larger than any of the latter. It is probable that the size of the relative sampling error of the total volume is well within the accuracy of present measurements of zone-to-zone movements and of predictions of trends.

Philadelphia Trips

In figure 19 and table 4 are presented data for trips between Philadelphia and the Pittsburgh metropolitan area for 3 years. The 1950 and 1952 trip information was obtained at the same locations in eastern Pennsylvania between Harrisburg and Philadelphia. The 1951 data were collected in western Pennsylvania just east of Pittsburgh in connection with a study of the western extension of the turnpike. The time of operation was the same for all three samples and represents a summer weekday. Though the 1951 information was obtained more than 150 miles from the nearest operation in the East, the Philadelphia-Pittsburgh trips were intercepted in both studies and the data should be comparable.

The similarity of the number of trips, the type of vehicle, the purpose of trips for pas-

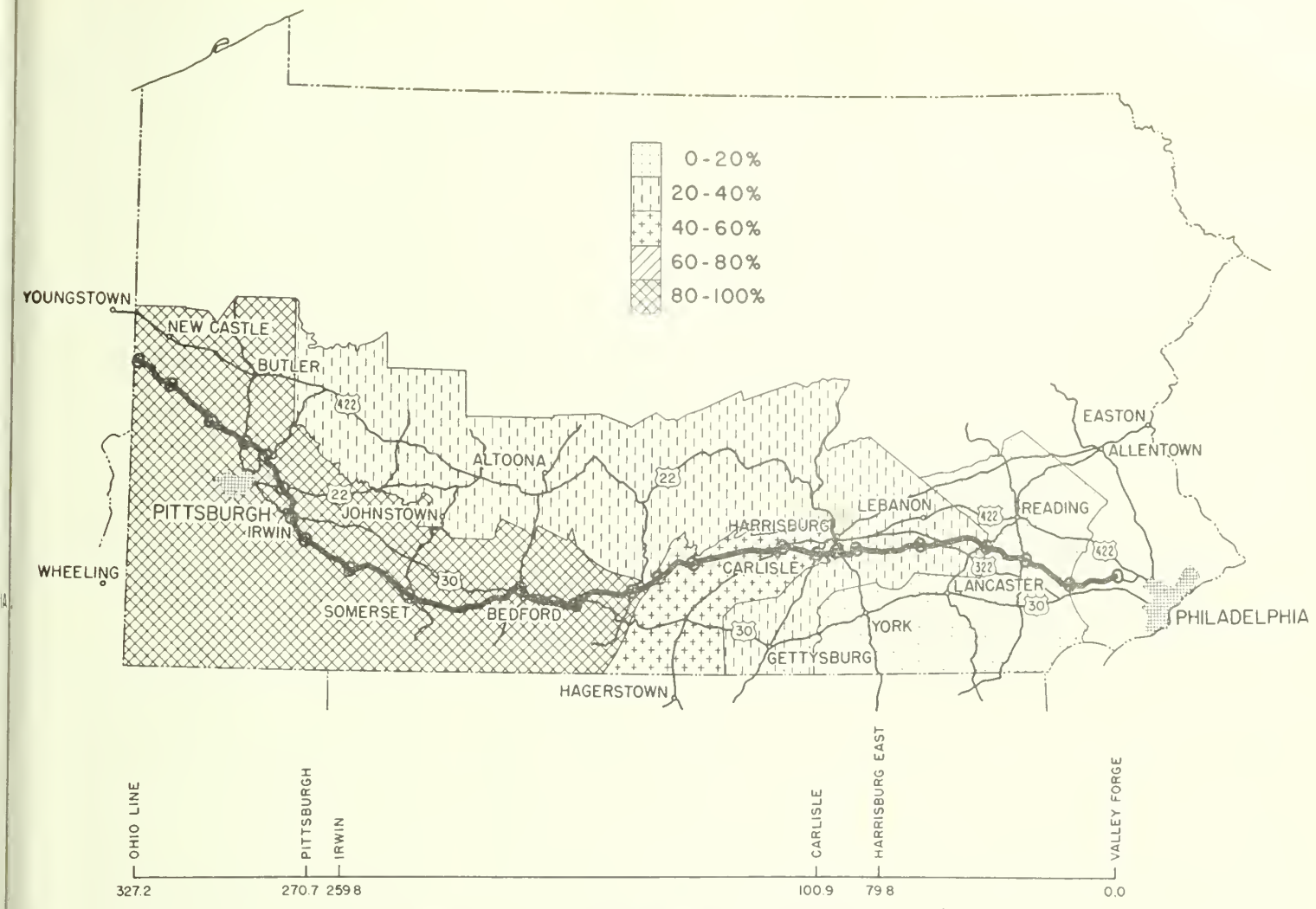

Figure 22.—Percentage of commercial vehicles using the turnpike between Philadelphia and areas along the route on an average summer weekday in 1952.

senger cars, and the percentage use of the turnpike in the several categories indicate the adequacy of the samples obtained in the three surveys. The number of passenger-car trips by purpose (work or business, social-recreation, vacation, and other) and the total trips for passenger cars and commercial vehicles with the latter divided into combinations and other commercial are shown in figure 19. The percentage of turnpike use by years for all purposes and for all types of vehicles is also indicated. A remarkable fact is that the average number of passenger-car trips between Philadelphia and Pittsburgh during the 3 years was almost exactly the same as the average number of commercial trips during the same period (228 and 225, respectively). However, passenger-car trips were increasing while commercial trips were decreasing. Also, about the same number of passenger-car and commercial-vehicle trips used the turnpike for the same 3 years (203 and 201, respectively). Trips for work or business and vacation increased steadily over the 3-year period, while those for social and recreation and for other purposes remained fairly constant. Turnpike usage by trips for work or business increased from 86 to 92 percent, but there was no apparent trend in turnpike usage by trips for the purposes in other categories. The high percentage use of the turnpike by passenger-car and commercial-vehicle trips is evidently due to the very great saving in time for both passenger cars and trucks. There is a large saving of fuel for trucks, particularly on the original section of the Pennsylvania Turnpike between Irwin and Carlisle.

The percentages of passenger-car trips between Philadelphia and Pittsburgh made for each purpose were similar for 1950, 1951, and 1952. Trips for work or business were 47, 46, and 49 percent of the total for each of the 3 years, respectively; for social-recreation 29, 28, 23; for vacation 22, 26, 26; and for other purposes 2, 0.5, and 2.

In figure 20 and table 4 are presented data showing the differences in turnpike usage for trips between Harrisburg and Philadelphia as compared with trips between Pittsburgh and Philadelphia and Pittsburgh and Harrisburg. Vehicles in the latter two categories use the turnpike to a considerably greater degree than those between Harrisburg and Philadelphia. The Pittsburgh-Philadelphia trips are shown for 1950, 1951, and 1952; the Pittsburgh-Harrisburg trips are shown for 1951 only, because these trips were not intercepted in the 1950 and 1952 surveys which were confined to the eastern extension; and the Harrisburg-Philadelphia trips are shown for 1952 only, because this section of the turnpike was not open in 1950 when the first survey on the eastern extension was made.

While 36 percent of all truck trips were made on the turnpike between Harrisburg and Philadelphia in 1952, only 23 percent of the combination vehicles used it. The reason for the low turnpike usage by these large vehicles has not been determined.

In 1951, 49 percent of the total trips between Pittsburgh and Philadelphia were made by commercial vehicles while only 18 percent were by commercial vehicles between Pittsburgh and Harrisburg. In 1952, 46 percent of the trips between Pittsburgh and Philadelphia were by commercial vehicles compared with only 21 percent for the Harrisburg-Philadelphia trips.

It was found that there was greater turnpike usage between Harrisburg and Philadelphia than between Harrisburg and the areas around New York City and New England. This was expected since a part of the traffic used the shorter routes via U. S. 22, going northeast from Harrisburg rather than the longer one via the turnpike and Philadelphia.

Figure 21 shows the relative passenger-car turnpike usage by trips between certain areas along the turnpike and Philadelphia in 1952. The areas are marked by different types of shading, so drawn that the usage of the turnpike by passenger-car trips between each area

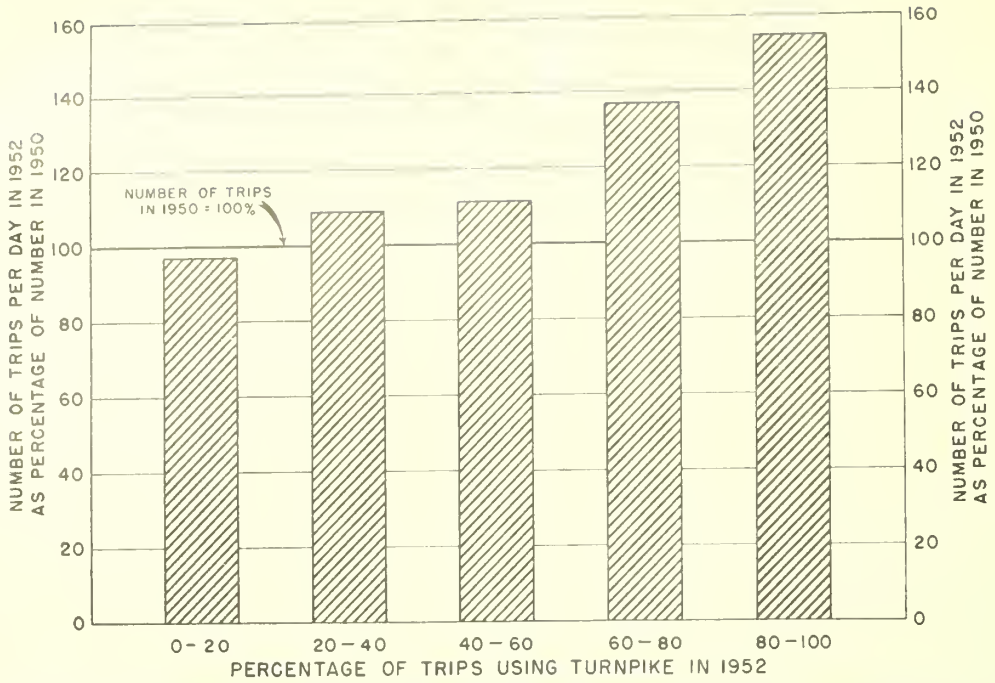
NUMBER OF TRIPS PER DAY IN 1952 AS PERCENTAGE OF NUMBER IN 1950
NUMBER OF TRIPS IN 1950 = 100%
NUMBER OF TRIPS PER DAY IN 1952 AS PERCENTAGE OF NUMBER IN 1950
0-20
20-40
40-60
60-80
80-100
PERCENTAGE OF TRIPS USING TURNPIKE IN 1952

Figure 23.—Percentage change, 1950–52, in passenger-car trips between Philadelphia and areas of varying turnpike usage.

and Philadelphia falls within a specified percentage range.

Eighty to one hundred percent of the passenger-car trips with origin and destination in the area west of a point midway between Carlisle and Bedford were made on the turnpike. This area extends upward across U. S. 422 just west of Johnstown. Of the trips between Pittsburgh and Philadelphia, 89 percent were made on the turnpike.

There is another area of 80- to 100-percent usage extending westward from Harrisburg and including Carlisle. For passenger cars, 60 to 80 percent of the vehicles traveling between Philadelphia and areas along U. S. 22 from Harrisburg to Johnstown used the turnpike. The area where the usage was less than 20 percent extends from the Philadelphia end of the turnpike westward for about 40 miles and farther south, almost 100 miles westward along U. S. 30.

Figure 22 shows the relative turnpike use by commercial-vehicle trips in and out of Philadelphia in the same manner as passenger cars are shown in figure 21. The area where the turnpike usage was less than 20 percent was the same as that for passenger cars. The percentage of turnpike use for commercial vehicles in this area was very low in most cases, being 5 percent for Reading, 5 percent for Lancaster, and only 1 percent for York. Between Harrisburg and Philadelphia, 36 percent of the commercial vehicles used the turnpike compared with 76 percent of the passenger cars. The area to and from which 20 to 40 percent of the Philadelphia trips were over the turnpike includes Harrisburg and extends westward to the north along the route of U. S. 22 and U. S. 422 to a point north of Pittsburgh. This is in contrast with 60- to 80-percent usage by passenger cars for this area. South of the turnpike from Gettysburg westward, Philadelphia trips with origin and destination close to U. S. 30 were made via the turnpike to a rapidly increasing degree. The 80- to 100-percent area is reached at a point about half way between Carlisle and Bedford. Beyond that point over 80 percent of the commercial trips in the area served by U. S. 30 as the principal free route were made on the turnpike.

The high usage of the Pennsylvania Turnpike by commercial vehicles is unique in turnpike experience and is due principally to the very steep grades and sharp curves on U. S. 30.

Traffic Generation

Figure 23 shows the growth in the numbe of trips between Philadelphia and the differen areas shown in figure 21 from 1950, before the turnpike was opened, to 1952 after it was opened. This comparison is for summe weekday trips. The 100-percent line on the chart represents the number of trips betwee the different areas and Philadelphia the yea before the turnpike was opened. For the area in which less than 20 percent of the Phila delphia trips were made on the turnpike, there was a 3-percent decline in the number of trips from 1950 to 1952. For the area in which 20 to 40 percent of the vehicles used the turnpike there was a 9-percent increase; for the 40- to 60-percent area, a 11-percent increase; for the 60- to 80-percent area, a 37-percent increase; and for the 80- to 100-percent area, a 55-percent increase. Thus, the higher the percentage use of the turnpike by trips to and from Philadelphia for any area the greater was the growth in the number of these trips after the turnpike was opened, indicating definitely that the increase was caused by the existence of the turnpike.

Figure 24 shows the percentage increase in the number of trips between Philadelphia and certain cities along the turnpike route after the turnpike was opened. For Reading, where the percentage use of the turnpike was 14 percent as indicated by the figure below the bar there was only a 2-percent increase in the number of Philadelphia trips, as indicated by the height of the bar; for Harrisburg where the usage was 76 percent, there was a 53-percent increase in the number of trips to and from Philadelphia. For Carlisle and the intervening area west of Harrisburg, where there was an

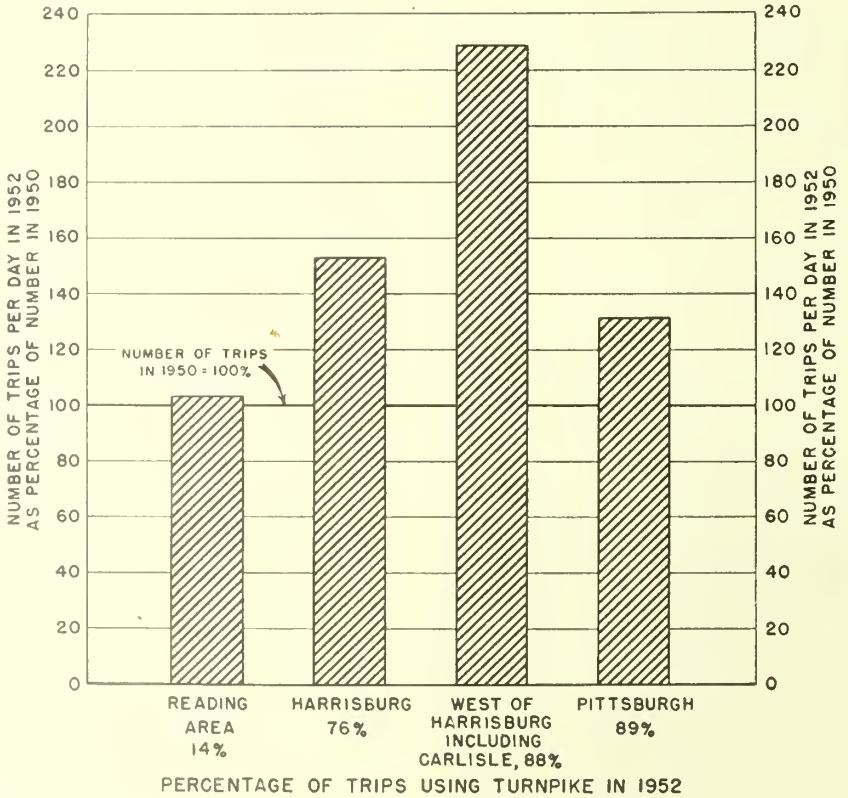
NUMBER OF TRIPS PER DAY IN 1952 AS PERCENTAGE OF NUMBER IN 1950
NUMBER OF TRIPS IN 1950 = 100%
NUMBER OF TRIPS PER DAY IN 1952 AS PERCENTAGE OF NUMBER IN 1950
READING AREA 14%
HARRISBURG 76%
WEST OF HARRISBURG INCLUDING CARLISLE, 88%
PITTSBURGH 89%
PERCENTAGE OF TRIPS USING TURNPIKE IN 1952

Figure 24.—Percentage change, 1950–52, in passenger-car trips between Philadelphia and cities along the turnpike.

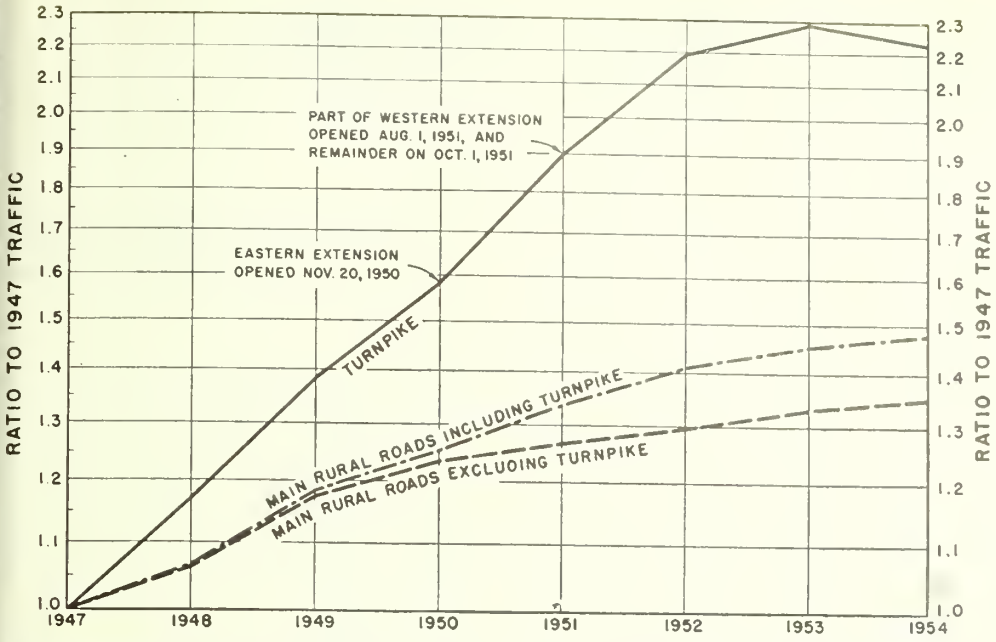

Figure 25.—Traffic growth on the original section of the turnpike (Irwin–Carlisle) compared with that on main rural roads in Pennsylvania, 1947–54.

88-percent usage of the turnpike for Philadelphia trips, the increase in the number of trips was 129 percent. This area is at the western end of the eastern extension of the Pennsylvania Turnpike and would, therefore, benefit more proportionately by the opening of the turnpike than any other area.

A number of these trips were undoubtedly new trips which would not have been made had not the turnpike been built, but some of them also were probably diverted from public carriers. Trips between Philadelphia and Pittsburgh have benefited by the existence of the original section of the Pennsylvania Turnpike from Irwin to Carlisle for many years, and the opening of the eastern extension was not so important, relatively, for these trips as it was for the trips between Carlisle and Philadelphia. For this reason, the percentage increase in the number of trips from Pittsburgh to Philadelphia after the opening of the eastern extension of the turnpike was only 31 percent.

From traffic counts before and after the opening of the turnpike, an attempt was made to estimate the total traffic generated between interchanges 19 (Harrisburg East) and 20 (Lebanon). It would appear that about 40 percent of the traffic on the turnpike between these interchanges was composed of new trips induced by the facility. However, some of these trips may have been diverted from routes so far away that they were not included in the study. In attempting to estimate traffic generation from counts alone, this uncertainty is always present.

Turnpike Traffic Growth

Figure 25 shows the growth from 1947 to 1954 of traffic on the original section of the Pennsylvania Turnpike between Irwin and Carlisle compared with that of other main rural roads in Pennsylvania, and all main rural roads including the eastern and western extensions of the turnpike during this same period. This figure is plotted on semilogarithmic paper, which has the characteristic of presenting equal rates of growth as parallel lines regardless of the height of the curve on the chart. Throughout the period from 1947 to 1952 the turnpike curve has a steeper slope than the curve for the other roads, indicating a more rapid growth. From 1952 to 1953 the two curves are very nearly parallel, and from 1953 to 1954 the turnpike curve drops, while the curve for other main rural roads continues to rise. This drop in traffic, 2.5 percent, on the old section of the turnpike in 1954 was largely due to a decline in truck usage of 6.4 percent, since passenger-car usage remained practically the same. Probably this is related to general economic conditions, there being a slight decline in the Gross National Product from 1953 to 1954.

The old section of the turnpike had been in operation for 7 years in 1947 so the more rapid growth of the turnpike traffic up to 1952 does not represent generation occuring soon after the opening of a new facility of this kind. There is no way of knowing whether it represents continuing generation or increasing diversion. The very steep slope of the turnpike curve from 1950 to 1952 undoubtedly shows the effect of the opening of the eastern and western extensions. The former was opened in November 1950 and the latter, partly in August and partly in December 1951. This is a good illustration of the effect of turnpike extensions and connections on traffic using the old section.

Figure 26 shows the growth of traffic on the old section of the Pennsylvania Turnpike throughout its life compared with that on other important rural roads in Pennsylvania, and on all rural roads including all sections of the turnpike. During the war years, traffic on the turnpike decreased much more rapidly than that on the other roads, but after 1944, the last full year of the war, it made a rapid recovery, and in 1954 it was 180 percent higher than in 1941, whereas traffic on other principal roads in Pennsylvania was only 26 percent, and including the turnpike, only 39 percent higher than in 1941. This exceptionally rapid growth of the Pennsylvania Turnpike traffic is probably due to unique features of this particular project and it would certainly be unwise to assume that such growth would be duplicated on other projects.

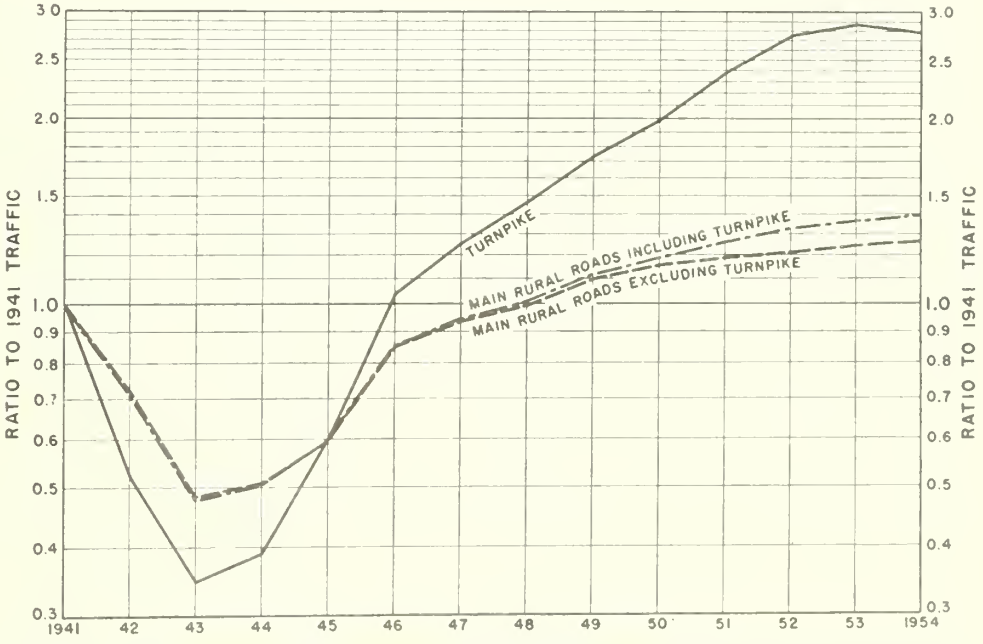

Figure 26.—Traffic growth on the original section of the turnpike (Irwin–Carlisle) compared with that on main rural roads in Pennsylvania, 1941–54.

Traffic Usage of Maine Turnpike

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by GLENN E. BROKKE
Highway Transport Research Engineer

The Maine Turnpike was opened to traffic on December 13, 1947, as an alternate route to U. S. 1 between Kittery and Portland, Maine. Traffic was interviewed both before and after construction and continuous volume counts have been available since 1947.

This article shows the traffic growth, the State of registration of the vehicles, the repetitive frequency of trips on both highways, and the effect of these variables on the proportion of traffic using the turnpike. An empirical curve of passenger-car traffic diverted to the toll road on the basis of time ratio is established and compared with similar curves for the Pennsylvania Turnpike and a free limited-access facility.

The effect of increasing the toll rate from $0.50 to $0.60 and subsequently to $0.75 for a full-length trip is investigated and related to the amount of traffic using the turnpike. The amount of traffic attracted by the construction of the new facility beyond that anticipated from normal increase is determined, and the distribution of the excess increase between the old road and the turnpike is reported.

THE Maine Turnpike, a 42.2-mile toll facility, provides an alternate route of travel to U. S. 1 between Portsmouth, N. H., and Portland, Maine, as shown in figure 1. The turnpike is a four-lane divided highway with full access control as compared to U. S. 1, a basically three-lane roadway with unrestricted access.

The local area directly served by the route has a population of about 64,000, which is an average density of about 200 persons per square mile. In addition, this local area borders the Atlantic Ocean and provides a strong attraction for recreation and vacation traffic during the summer months.

Portland, which is located immediately north of the turnpike, has an urban area population of about 115,000; and Portsmouth, which is immediately south of the turnpike, has a population of about 19,000. Between these cities the principal sources of local traffic are to the east of U. S. 1, while the turnpike is located to the west of U. S. 1. Thus, the use of the turnpike results in excess travel for a majority of the local trips.

Access to the turnpike is provided at four intermediate toll stations as well as at each terminal station. A fifth intermediate toll station between the Saco interchange and the north terminus is operated during the summer months.

Previous Studies

Origin and destination surveys were made in August and October of 1947 and 1948 and in August of 1950. Partial results of these surveys were presented at a meeting of the Highway Research Board.[1]

Data from both the Maine and Pennsylvania Turnpikes were combined in a paper presented at the annual meeting of the American Society of Civil Engineers in 1954.[2]

It is the primary purpose of this article to show the relative magnitude of the various factors that influence the amount of traffic on the Maine Turnpike. The data reflect conditions as they exist in this area and, consequently, the conclusions will not necessarily be applicable in areas of substantially different characteristics.

[1] *Proportionate use of Maine Turnpike by traffic through Portsmouth-Portland corridor*, by J C Carpenter. Proceedings of the Highway Research Board, vol. 32, 1953, pp. 452–463.

[2] *Traffic diversion to toll roads*, by John T. Lynch. Proceedings of the American Society of Civil Engineers, vol. 81, separate No. 702, June 1955.

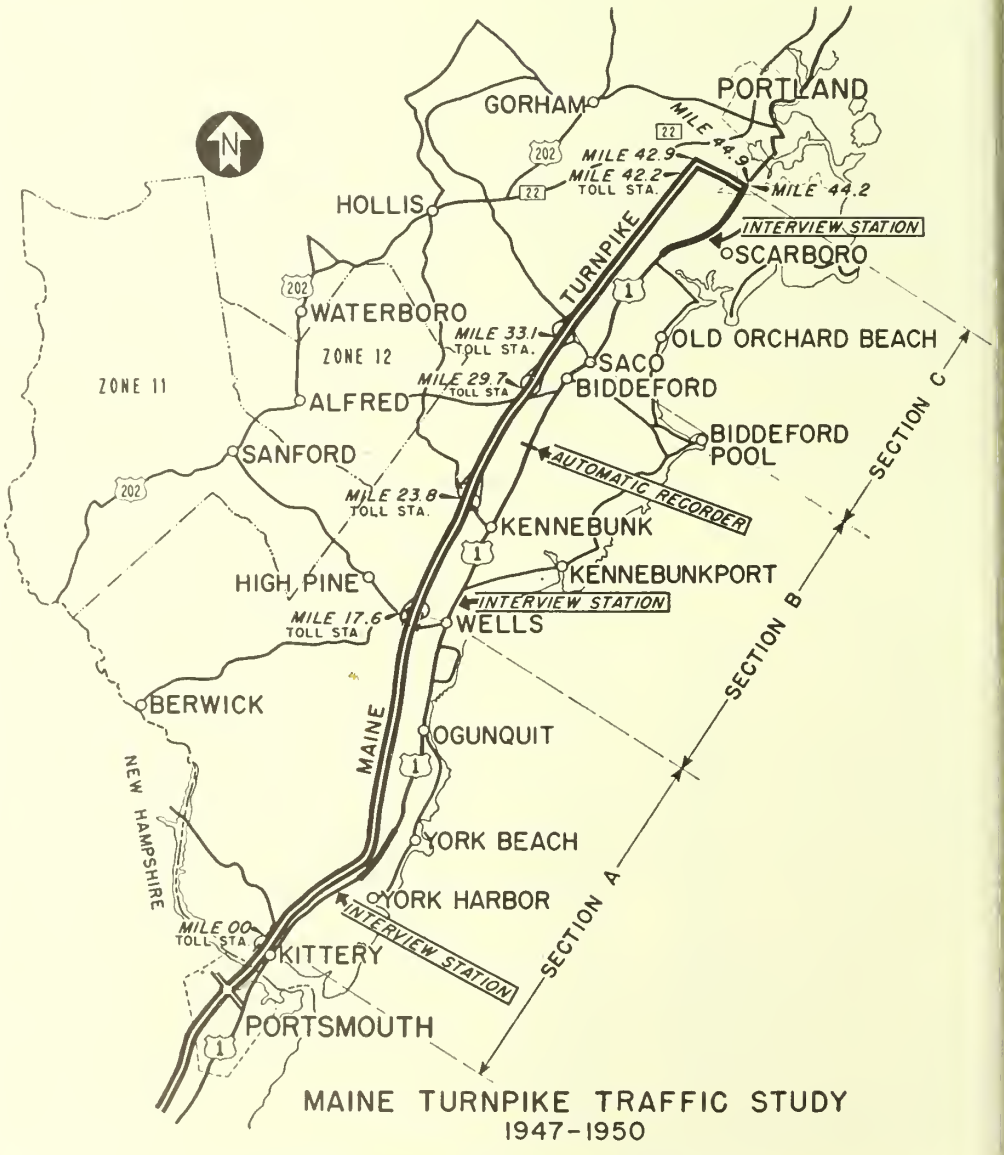

Figure 1.—Sketch of the Maine Turnpike corridor.

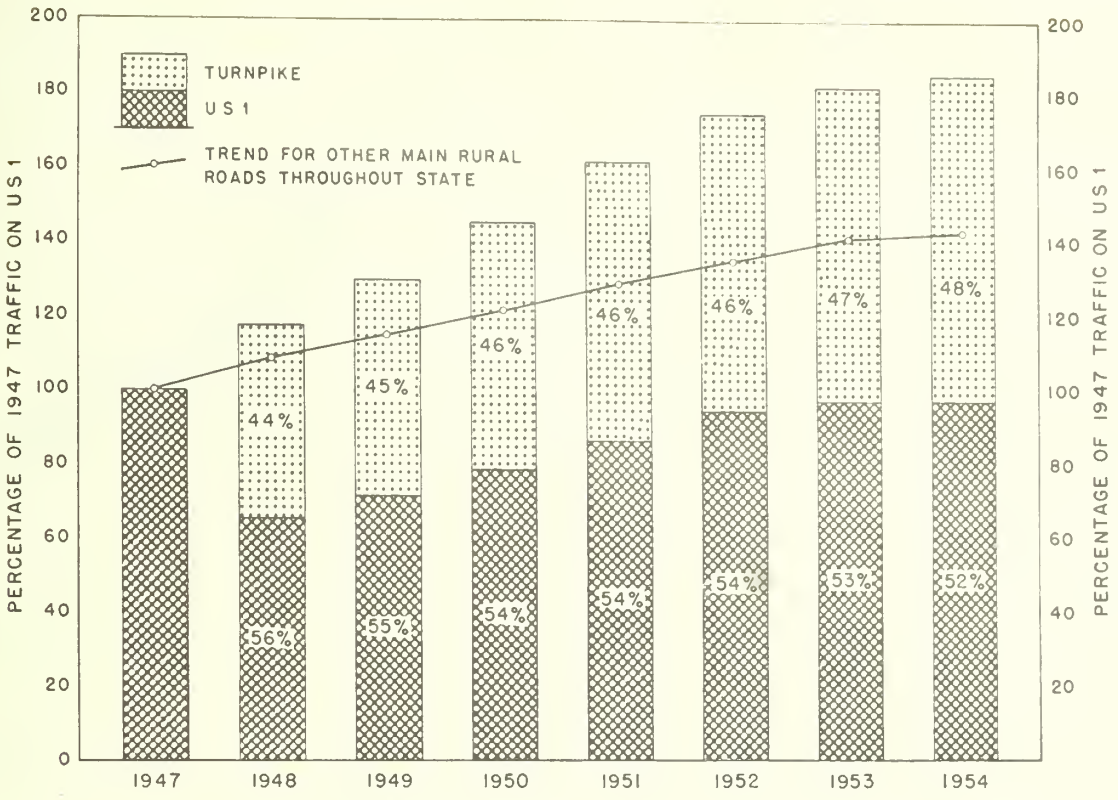

Figure 2.—Traffic using the Maine Turnpike as related to parallel route U. S. 1 at Mile 29 for years 1948–54, as compared with 1947.

Conclusions

To determine accurately the amount of traffic that will be attracted to a new facility, it is necessary to obtain information concerning the origin and destination of trips and relative speeds on the existing highways as compared to the new facility. It has been found that the most accurate single measure of traffic diversion is time ratio; that is, the time required to make a trip on the improved facility divided by the time required to make it by the existing route.

For preliminary purposes, rough estimates of the proportion of the traffic which would use a new facility can be made on the basis of less comprehensive field information, particularly if most of the potential traffic consists of diversion from an adjacent parallel route. Formulae for relating both traffic volume and State of vehicle registration to turnpike usage are developed in this article.

These approximate measurements might serve to determine whether a proposed project has sufficient merit to justify further examination by means of a study of the origins and destinations of the potential traffic.

The traffic served by a new limited-access facility and the old parallel highway is the existing traffic multiplied by a normal increase factor and further multiplied by a generation factor. In the case of the Maine Turnpike and parallel U. S. 1, the generation factor increased at approximately 6 percent a year for 5 years, after which it remained constant at approximately 1.30.

In the case of Maine, origin and destination data obtained from screen lines at both ends of the proposed turnpike would have accounted for 99 percent of the traffic using the turnpike.

Table 1.—Average daily traffic volume for the Maine Turnpike and parallel route U. S. 1 for the years 1948–54

Year	Average daily traffic for—			Percent of traffic using turnpike
	U. S. 1	Maine turnpike	Total	
1948	4,194	3,321	7,515	44.2
1949	4,577	3,719	8,296	44.8
1950	5,033	4,252	9,285	45.8
1951	5,551	4,809	10,360	46.4
1952	6,039	5,144	11,183	46.0
1953	6,222	5,438	11,660	46.6
1954	6,229	5,653	11,882	47.6

From the origin-destination data, the present and estimated future trips can probably be assigned to a toll road from the Maine Turnpike or Pennsylvania Turnpike curve or an interpolation between the curves.

The time-ratio curves appear to be applicable for toll rates up to at least 1¾ cents per passenger-car mile.

Traffic Growth

Figure 2 shows the trend in traffic on U. S. 1 and the Maine Turnpike from 1947 (the last year before the turnpike was opened) through 1954. The U. S. 1 traffic data used in this comparison were recorded near the middle of the Maine Turnpike area where the volume was lowest, in order to eliminate as much as possible the effect of local traffic which could not conveniently use the toll road.

In figure 2, the 1947 traffic volume on U. S. 1 is assigned the value of 100. The line sloping upward on the chart represents the trend of traffic on other important routes throughout Maine in relation to 1947 traffic. The height of the individual bars represents the total amount of traffic in each year. The shading of the bars denotes the portion of the traffic that used U. S. 1 and the turnpike. Figures within the shaded portions indicate the percentage of total traffic that used each of these facilities.

Traffic Diversion Related to Corridor Volume

Figure 2 indicates the most simple measure of traffic diversion. If U. S. 1 and the turnpike are defined as the corridor, then for each value of annual traffic volume in the corridor, a percentage diversion to the Maine Turnpike can be determined. Table 1 shows the average daily traffic volumes for the Maine Turnpike and parallel route U. S. 1, and the percentage of traffic using the turnpike for each of the years, 1948–54. The data essentially represent a straight line indicating a basic diversion of 39 percent to the turnpike, plus 1 percent for every 1,400 average daily traffic in the corridor. This relation may be expressed as follows:

$$\text{Percentage diversion to turnpike} = 39 + \frac{\text{ADT}}{1{,}400}$$

In order to compare the Maine Turnpike with other limited-access highways, it is desirable to take into account the surfaced width of the highway from which traffic is being diverted. U. S. 1 has an average width of 32.8 feet. Thus, the figure 1,400 for average daily traffic can be interpreted as approxi-

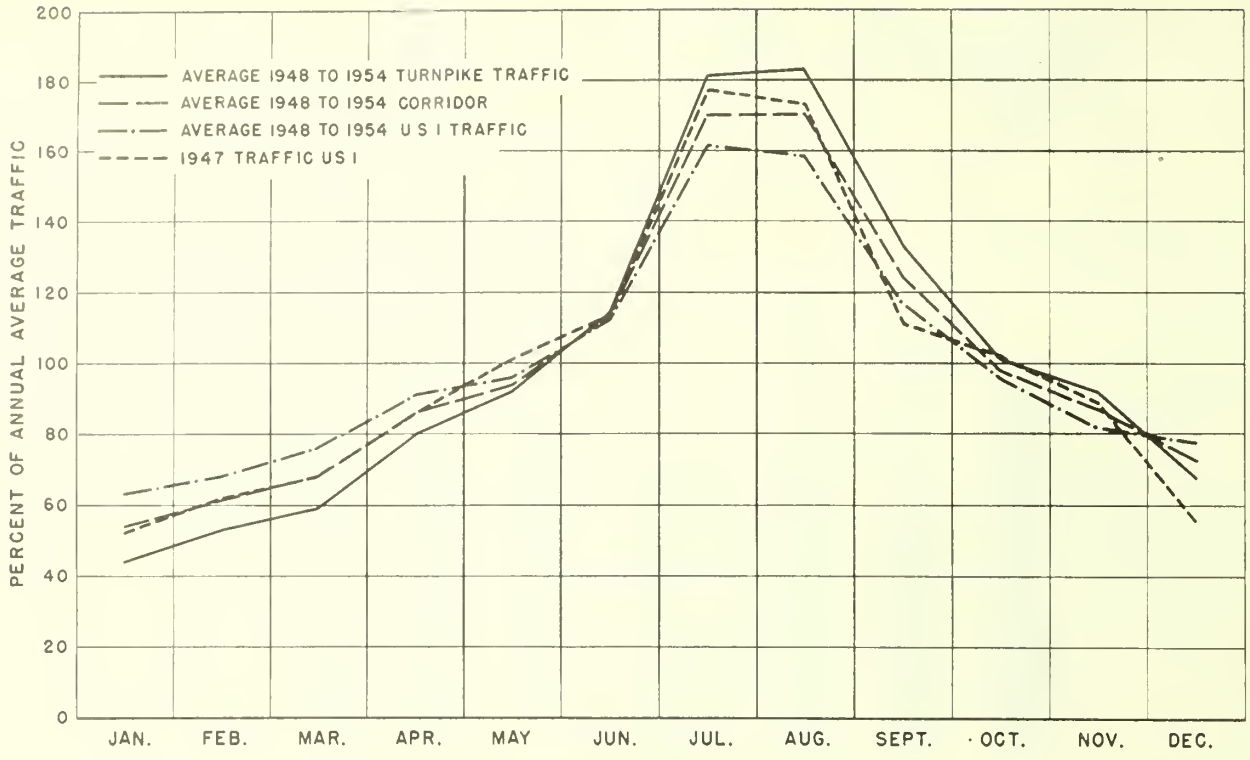

Figure 3.—Monthly variations in the traffic flow.

mately 425 ADT per 10-foot lane. The percentage diversion on a volume basis can therefore be more generally stated as:

$$\text{Percentage diversion to turnpike} = 39 + \frac{\text{ADT per 10-foot lane}}{425}$$

Inasmuch as the Maine area attracts resort travel, the summer season has a much higher traffic volume than the remainder of the year. The magnitude of this variation is shown in figure 3.

It is of interest to note that generally during those months in which the corridor is carrying more than the annual average volume, the turnpike is carrying relatively more traffic than U. S. 1. Conversely during the months of less than annual average volume, U. S. 1 is carrying relatively more traffic. The seasonal fluctuations in traffic on U. S. 1 have been reduced somewhat by the construction of the turnpike, because the latter provides needed relief during peak flow.

The percentage of the total traffic that is diverted to the turnpike for each month of the year was computed, thereby establishing a percentage diversion for a wide range of traffic volumes. The individual months for the 7 years following the turnpike opening provided 84 plotting points for determining a curve relating percentage diversion to traffic volume. Naturally there was some scatter to these points, but the use of a series of moving averages resulted in a very smooth curve. The plotted points and the curve are shown in figure 4. In addition, figure 4 shows the line, previously discussed, representing the percentage diversion for the average daily traffic volumes in the different years. The plotted points from which this line was established are also shown.

The curve for the monthly traffic volumes departs substantially from the straight line for the annual average figures, although the curves are reasonably close for the corridor volumes in which they overlap. The reason that the curve for monthly traffic volumes diverges downward from the line for annual traffic volumes is probably that low volume counts occur during the winter when travel is predominantly local, while the high volume months are in the summer when there is a substantial volume of out-of-State traffic. Thus, it is likely that the percentage of foreign vehicles (out-of-State) influences the percentage diversion to the turnpike.

As a matter of interest, the diversion percentages for the first 2 years' operation of the Oklahoma Turnpike are also plotted in figure 4. These two points are somewhat above the Maine Turnpike curve which indicates a larger diversion percentage. Part of this excess diversion is probably attributable to attraction from more distant routes. However, figure 4 indicates that a percentage diversion based on volume does provide a rough approximation of the traffic on a toll road.

States of Registration

Table 2 shows the percentage of passenger cars using both U. S. 1 and the Maine Turnpike according to the States in which the vehicles are registered. From this table it is evident that foreign passenger cars use the turnpike in greater proportion than do local passenger cars. It is also evident that there is a material difference in usage by vehicles registered in the various States.

Passenger cars registered in New Hampshire show less tendency to divert to the turnpike than do passenger cars registered in Massachusetts. New Hampshire is adjacent to the south end of the turnpike while Massachusetts is centered approximately 80 miles south of the Maine Turnpike. However, vehicles registered in States more distant than Massachusetts have only a slightly higher tendency to divert and, as a matter of fact, passenger cars registered in States more than 500 miles

Table 2.—Passenger-car traffic on both U. S. 1 and the Maine Turnpike according to State of registration

State of registration	August 1948		October 1948 [1]		August 1950	
	Percent of corridor traffic	Percent using turnpike	Percent of corridor traffic	Percent using turnpike	Percent of corridor traffic	Percent using turnpike
Maine	33.4	32	59.7	32	37.7	38
Massachusetts	31.5	62	20.1	75	32.0	62
New York	8.2	55	2.8	78	6.4	54
New Hampshire	6.6	35	5.6	53	6.5	42
Connecticut	3.9	66	2.9	80	3.6	67
New Jersey	3.1	55	1.3	80	2.6	60
Pennsylvania	2.3	64	1.0	73	2.0	57
Rhode Island	2.2	60	1.3	79	2.3	59
Vermont	.5	37	.3	47	.5	38
All other	8.3	52	5.0	64	6.4	48
Total	100.0	49	100.0	48	100.0	50

[1] Weekday traffic only.

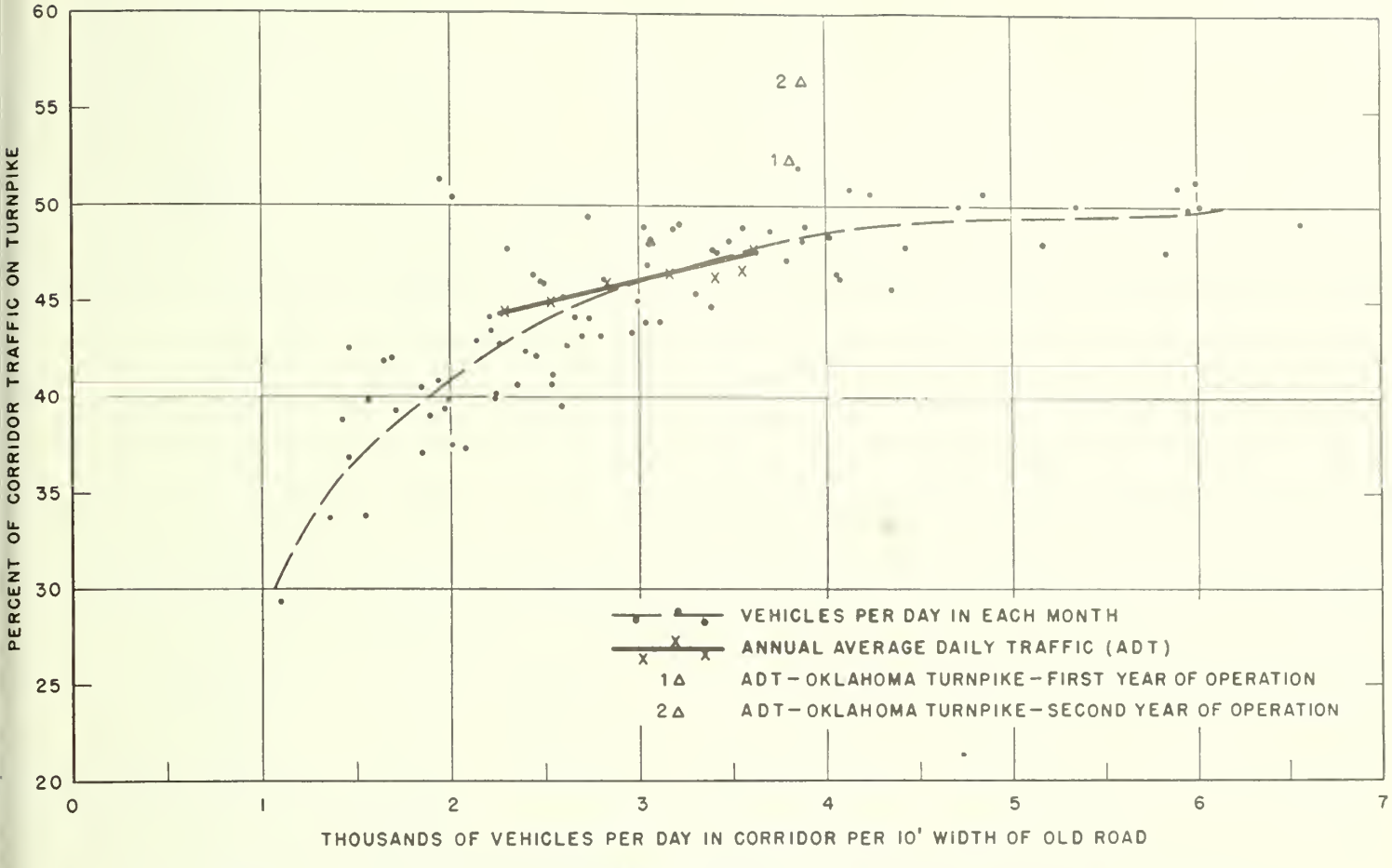

Figure 4.—Percentage of traffic diverted to the Maine Turnpike as related to total traffic volume in the corridor.

away appear to have slightly less tendency to divert than do vehicles registered in Massachusetts.

In addition to distance, the location of the State relative to the general axis of the turnpike appears to influence the diversion percentage. For example, Vermont is about the same distance from the turnpike as Connecticut, yet approximately one-half the percentage of Vermont cars use the turnpike as compared with Connecticut cars. In this connection it is noted that Vermont is located approximately at right angles to the turnpike while Connecticut is almost in line with the turnpike.

Finally, it is also evident that the diversion percentage of foreign traffic is dependent on the season of the year. Presumably many of the foreign cars, particularly from the more distant States, spend an appreciable amount of time in the turnpike area and become, in effect, local cars for the duration of their visit.

A mathematical expression which takes into account the factors mentioned and which approximates the diversion to the turnpike is as follows:

During resort season:

$$\text{Diversion} = \tfrac{1}{3} + \tfrac{1}{3} K \cos A$$

Where:

$K = 0$ for State of Maine.

$K = \dfrac{\text{Distance}}{100}$ for distances between 0 and 100 miles.

$K = 1$ for States more distant than 100 miles.

A = Angle to the center of population of the State from general axis of turnpike.

During other times:

$$\text{Diversion} = \tfrac{1}{3} + \tfrac{1}{2} K \cos A$$

Where:

K and A have the same significance as above.

While this formula may apply only in the case of the special situation of the Maine Turnpike, it may be worthwhile to test it in other areas to see whether the formula, or a modification of it, would be useful in making preliminary traffic estimates to determine whether a proposed project might be worthy of further investigation.

Trip Frequency

As traffic is stopped and interviewed, a number of other characteristics of the traffic can be examined in more detail. One of the general elements that will increase the understanding of traffic behavior is that of trip frequency. Motorists who make a trip through an area every day will likely be acquainted with the location and condition of the various alternate routes. The infrequent user is more likely to follow route markings or to rely on general conceptions of the desirability of various routes. The trip frequency of traffic in the corridor, as determined from the interviews, is shown in table 3.

It is of considerable significance that during the summer season well over half the traffic was making the trip annually or less frequently. Even in October these infrequent trips account for approximately one-third of the traffic. Generally speaking, a larger percentage of vehicles making infrequent trips would be expected to use the turnpike than would those making trips on a more frequent basis. The actual percentages are shown in table 4.

Table 3.—Trip frequency of traffic in the Maine Turnpike corridor

Trip frequency	1947		1948		August 1950
	August	October [1]	August	October [1]	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Daily	15	23	10	18	7
Weekly	17	26	16	23	19
Montbly	10	17	12	18	12
Yearly	35	24	40	26	45
Less than once yearly	23	10	22	15	17
Total	100	100	100	100	100

[1] Weekdays only.

Table 4.—Percentage of corridor traffic using Maine Turnpike according to trip frequency

Trip frequency	August 1948	October 1948	August 1950
Daily	11	15	12
Weekly	31	34	29
Monthly	57	62	51
Yearly	61	80	65
Less than once yearly	54	33	49
Average	49	48	50

It is noted that the percentage use varies with frequency about as expected except for those trips which are less frequent than annual. Trip frequency, however, does provide a rather sensitive indication of turnpike use.

Time Ratio Studies

With origin and destination data available, it is possible to establish the traffic diversion to the turnpike based on distance and time factors. The distance between zones of origin and destination was measured from maps. Travel time on U. S. 1 and the turnpike was obtained from field measurements. A time-ratio value was computed as time required to drive from origin to destination via the turnpike divided by the time required via U. S. 1, except for trips with either origin or destination beyond the limits of the turnpike. For these longer trips, the time was computed from a point near the turnpike where the motorist must make a choice of either U. S. 1 or the turnpike. The time ratio was computed from this point of choice. Figure 5 shows the diversion of passenger cars based on time ratio.

Data for trips in August of 1948 and 1950 were used in establishing the time-ratio curve. From the 2-year traffic data, a measure of the reliability can be obtained by comparing the diversion in each year with the average diversion of both years.

The area served by the Maine Turnpike was divided into 31 zones. The total number of trips with origin or destination in each of these zones, together with the percentage of trips made on the turnpike, were established for 1948 and 1950. The diversion to the turnpike of trips for each zone varied from less than 3 percent in the local area to more than 70 percent in areas of surrounding States.

Assuming that the average of 1948 and 1950 data represents the most probable value of diversion, the standard deviation of diversion percentages for each of the 31 zones for each year and for the time-ratio curve is as follows:

	Standard deviation percent
1948 data	4.5
1950 data	3.4
Time–ratio curve	8.0

In examining each of the individual zones, it was noted that vehicles on trips from zones 11 and 12 (fig. 1) actually divert to the turnpike at a much higher percentage than anticipated from the time-ratio curve. A study of the highway network of the area indicates the reason for this apparent high diversion. Interviews were conducted on only U. S. 1 and the turnpike. Vehicles traveling between zones 11 or 12 and Portland (or farther north) can also use U. S. 202 and State Route 22. The turnpike can and probably does attract some of the trips from these routes thus increasing the number of trips on the turnpike. However, the trips remaining on U. S. 202 and State Route 22 are omitted from the sample. Since all of the diverted trips but only part of the total trips are included in the sample, the actual percentage diversion is understandably high.

Since the total volume of trips from zones 11 and 12 is small and the time-ratio curve is the result of the smoothed integration of all trips, the high diversion from these two zones does not materially affect the time-ratio curve. However, from a statistical standpoint, the trips from these zones contribute an inordinate amount of the standard deviation. If zones 11 and 12 are omitted, the standard deviation of the remaining 29 zones is 5.9 percent instead of the 8 percent for all zones.

Comparison With Other Time-Ratio Curves

While the time-ratio curve has an acceptable degree of statistical accuracy when used with Maine data, the question naturally arises as to how it compares with other studies made of time-ratio curves.[3] In the articles cited in the

[3] *The effect of travel time and distance on freeway usage*, by Darel L. Trueblood. PUBLIC ROADS, vol. 26, No. 12, Feb. 1952; also *Pennsylvania Turnpike traffic analysis*, by Daniel O'Flaherty. Pages 203–223 of this issue.

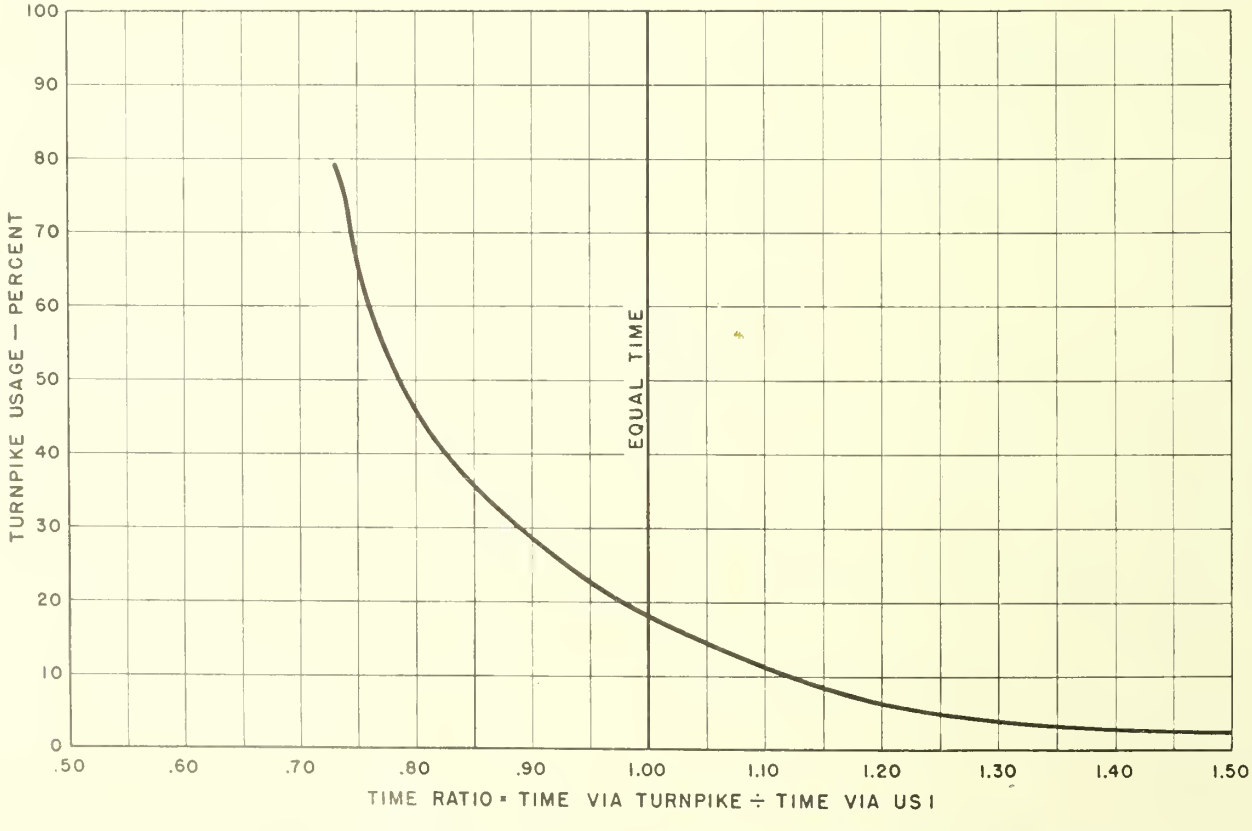

Figure 5.—Percentage of passenger cars using the Maine Turnpike in relation to the time ratio.

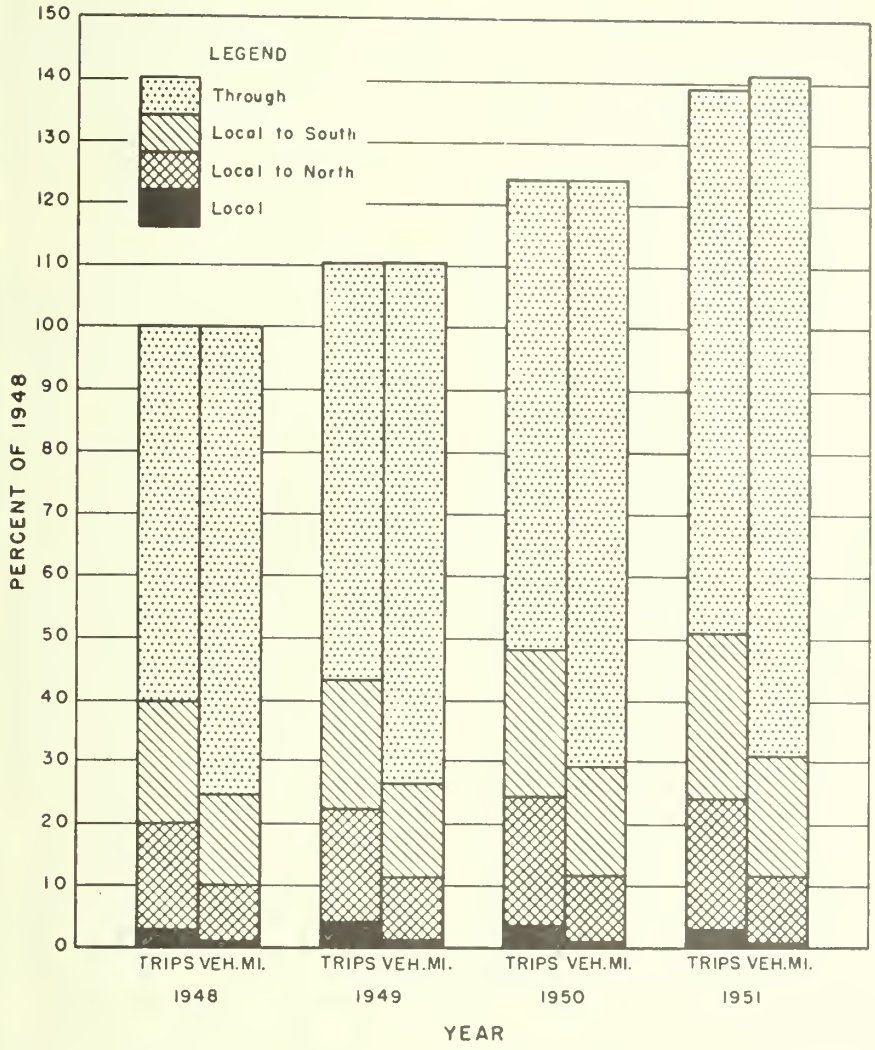

Figure 6.—Through and local passenger-car trips and vehicle-miles of travel on the Maine Turnpike for years 1949–51, as compared with 1948.

for the curve rotation is probably the effect of distance. The trips on the Pennsylvania Turnpike will average substantially longer than trips on the Maine Turnpike. Therefore, for equal time ratios other than 1.0, the time saved or lost would be much greater for the Pennsylvania than for the Maine Turnpike, tending to increase the attractiveness of the longer turnpike where the ratio is less than 1.0 and to decrease it where the ratio is greater than one.

A study of the rotation of the two curves indicates a number of features that should be kept in mind as further data become available.

1. The magnitude of time saved has an effect on the percentage diversion. At a constant time ratio, the greater the time saved the greater the diversion.

2. The magnitude of time lost also has an an effect on the percentage diversion. At a constant time ratio, the greater the time lost the less the diversion.

3. While the magnitude of time saved and time lost affects the diversion, the effect is less than that of time ratio. Thus, it is better to modify the time-ratio curves by considering the minutes saved or lost, than it would be to modify minutes saved or lost curves by time ratio.

4. The fact that the diversion is the same for a time ratio of 1.0 on both the Maine and Pennsylvania Turnpikes suggests that time ratio is a more vital factor than distance in traffic diversion, considering the fact that at this time ratio, the adverse distance would, on the average, be greater for the longer turnpike.

5. The Maine and Pennsylvania Turnpikes approximate the minimum and maximum length of turnpikes in the United States. Most turnpikes will fall into categories somewhere between these extremes.

footnote, it is found that the curves show greater diversion to the limited-access facilities. The Shirley Freeway curve (Shirley Memorial Highway in Virginia) if applied to the Maine data would result in 38 percent greater diversion to the turnpike than actually exists, illustrating primarily the greater attraction of traffic to a free road, though other factors are involved such as more points of access on the Shirley Highway, and the urban characteristics of the area through which the freeway passes as compared to the rural characteristics of the Maine Turnpike area.

Similarly the Pennsylvania Turnpike curve, though for a toll road, indicates an 18 percent greater diversion to the Maine Turnpike. It is noted, however, that the Maine and Pennsylvania curves have a striking similarity. Each of the curves show a diversion of about 18 percent of the traffic when the time ratio is 1.0. For higher time ratios, the Maine Turnpike curve indicates greater diversion than the Pennsylvania Turnpike curve and for lower time ratios, the Maine curve indicates less diversion than the Pennsylvania curve. Thus if the Maine curve is rotated about the point of 1.0 time ratio, it will practically coincide with the Pennsylvania curve. The reason

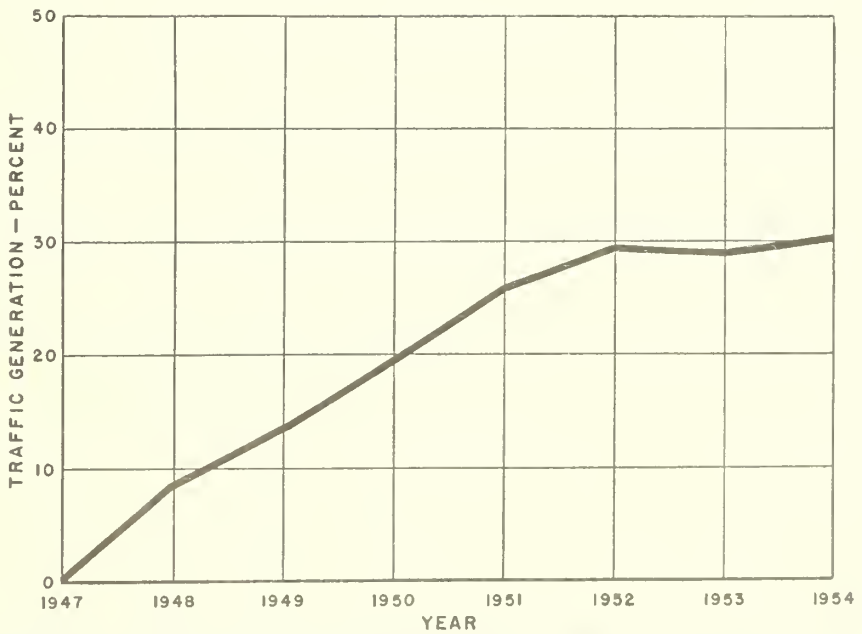

Figure 7.—Percentage of traffic generated in the Maine Turnpike corridor, 1947–54.

Table 5.—Traffic increase on U. S. 1 and the Maine Turnpike, 1948–54

Year	U. S. 1			Maine Turnpike			Total generated traffic	Percent generated on turnpike
	Actual count	Estimated normal increase	Generated traffic	Actual count	Estimated normal increase	Generated traffic		
1948	4,194			3,321				
1949	4,577	4,433	144	3,719	3,510	209	353	59
1950	5,033	4,701	332	4,252	3,723	529	861	61
1951	5,551	4,982	569	4,809	3,945	864	1,433	60
1952	6,039	5,234	805	5,144	4,145	999	1,804	55
1953	6,222	5,477	745	5,438	4,337	1,101	1,846	60
1954	6,229	5,532	697	5,653	4,380	1,273	1,970	65

Types of Trips Made on the Turnpike

Traffic using the Maine Turnpike in the years 1948 through 1951 was divided into four groups as follows:

Through trips.—Trips that were made between the two terminal interchanges.

Local south trips.—Trips made between an intermediate interchange and the south terminal interchange.

Local north trips.—Trips made between an intermediate interchange and the north terminal interchange.

Local trips.—Trips made between two intermediate interchanges.

Figure 6 shows the relative number of trips in each category and the contribution of each type of trip to the total vehicle-miles traveled on the turnpike. The total trips and vehicle-miles of travel in 1948 are shown as 100 percent, and the quantities in the succeeding years are related to the 1948 base year. It can be seen that through trips are not only the most frequent but contribute more than 75 percent of the total traffic in each of the 4 years.

The usefulness of figure 6 is primarily in determining the number of interview lines that would be required in estimating traffic on a facility comparable to the Maine Turnpike.

Interview lines at the north and south ends intercepted about 99 percent of the total traffic using the turnpike. A south line alone would account for 90 percent of the traffic while one at the north alone, would account for about 85 percent of the traffic.

Effect of Toll Rate

The Maine Turnpike offers a good opportunity to measure the effect of various toll rates. In May 1949, the toll was increased from 1.18 to 1.41 cents per passenger-car mile and in May 1952, the rate was further increased to 1.76 cents per passenger-car mile. If the increased toll has a deterrent effect, the percentage of traffic diverted to the toll road should be measurably less after the toll is increased.

Actually, the diversion changed only slightly with the increase in toll. In the 11 months subsequent to May 1949, the traffic diversion was only 0.9 percent less than the 11 months preceding May 1949. Similarly in the 11 months subsequent to May 1952, traffic diversion was only 1.9 percent less than in the 11 months preceding May 1952. From data appearing in an earlier part of this article, it was shown that diversion tends to increase as the total corridor volume increases. In the 11 months subsequent to May 1949, the average corridor traffic volume increased by 675 vehicles per day which corresponds to a 0.5-percent increase in diversion. Similarly the increased corridor volume subsequent to May 1952, was 1,150 vehicles per day corresponding to an 0.8-percent increase in diversion.

It, therefore, seems reasonable to assume that increasing the toll rate from 1.18 to 1.41 cents per passenger-car mile had a total effect of losing the 0.5 percent anticipated increase plus the 0.9 percent actual decrease for a total decrease of 1.4 percent in diversion of passenger cars. Similarly the increase in tolls from 1.41 to 1.76 cents per passenger-car mile had the effect of losing the 0.8 percent anticipated increase plus the 1.9 percent actual decrease or the total decrease of 2.7 percent. Combining the two figures, the total effect of increasing the toll rate from 1.18 to 1.76 cents per passenger-car mile was a 4.1-percent reduction in the turnpike percentage of passenger cars passing through the corridor, which amounts to an 8.2-percent decrease in the number of passenger cars using the turnpike. Since the toll rate was increased 50 percent and traffic only decreased 8.2 percent, the net result was a 38-percent increase in revenue.

It is, therefore, apparent that toll rates up to 1.76 cents per passenger-car mile will be paid in areas similar to the Maine Turnpike corridor without a significant change in the number of passenger cars using the toll road.

Traffic Generation

Figure 2 indicates that total traffic on both the turnpike and U. S. 1 has been increasing at a greater rate than for other important roads in the State. This excess increase is caused, for the most part, by traffic generation although the increase probably includes diversion from more distant routes, normal variation in traffic flow, and development traffic.

Assuming the increase on the other important rural roads represents the increase that could have been anticipated had not the turnpike been constructed, the excess increase in the volume of traffic in the Maine Turnpike corridor is shown in figure 7 as traffic generation. The percentage shown is the amount that normally increasing traffic would have to be further increased to equal the actual traffic volume. Figure 7 indicates that traffic in the corridor increased at a greater rate than normal for about 5 years, at which time it was about 30 percent larger than would have been anticipated from normal growth.

While figure 7 indicates traffic generation on both U. S. 1 and the turnpike, it does not provide any information showing which facility was used by the generated traffic. This can be approximated in another manner.

If the 1948 average daily traffic on U. S. 1 and the turnpike is increased at the same rate as other important roads in Maine, it will be found that the computed traffic is not as large as the actual traffic as shown in table 5.

From the table it can be seen that about 60 percent of the traffic generated since 1948 is attracted to the turnpike and about 40 percent to U. S. 1. It is of interest to note that U. S. 1 continued to increase at a rate greater than normal until 1953, after which it increased at a rate less than normal. The average daily traffic on U. S. 1 in 1953 and 1954 was practically constant at about 6,225 vehicles, which is approximately 1,900 vehicles per day per 10-foot lane—nearly the same as in 1947. This observation suggests that U. S. 1 has again reached a point of congestion where traffic seeks to avoid it.

New Publication

A recent report, *The Financing of Highways by Counties and Local Rural Governments, 1942–51*, is the second publication resulting from an extensive long-term study by the Bureau of Public Roads. The report presents a discussion, and detailed statistical data, concerning the financing of highways by the county and local rural governments during the 10-year period. Included is information for each year, by States, on county and local receipts, expenditures, and debt for rural highways. Comparisons with earlier years are included in the summary tables and charts.

The report was made possible by the collection of the basic data through the efforts of the State highway departments, the county and local governments, and the field offices of the Bureau of Public Roads. The analysis and presentation were the work of the Financial and Administrative Research Branch, Bureau of Public Roads.

This publication is a sequel to the first study report, *The Financing of Highways by Counties and Local Rural Governments, 1931–41*. The present report is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 75 cents a copy.

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PUBLICATIONS

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Bibliography on Highway Lighting (1937).

Bibliography on Highway Safety (1938)

Bibliography on Land Acquisition for Public Roads (1947).

Bibliography on Roadside Control (1949).

Express Highways in the United States: a Bibliography (1945).

Indexes to PUBLIC ROADS, volumes 17–19 and 23.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1955

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$24,105	$11,897	$6,316	285.9	$5,210	$2,904	64.0	$44,376	$22,551	726.3	$61,483	$31,771	1,076.2
Arizona	12,337	2,850	2,165	65.6	1,347	947	40.9	10,690	7,812	185.0	14,887	10,924	291.5
Arkansas	19,656	6,372	3,414	248.4	5,037	2,701	68.2	21,702	11,040	534.7	33,111	17,155	851.3
California	27,936	45,874	25,117	262.2	8,061	4,625	8.2	133,431	65,777	319.0	187,366	95,519	589.4
Colorado	20,814	8,948	4,997	182.4	1,888	1,202	18.4	19,577	10,523	188.6	30,413	16,722	389.4
Connecticut	23,164	1,140	570	6.0	506	250	.9	9,309	4,602	12.3	10,955	5,422	19.2
Delaware	6,951	2,546	1,288	15.8	400	207	13.0	8,106	4,395	23.1	11,052	5,890	51.9
Florida	18,338	20,742	10,773	345.0	10,242	5,158	72.7	24,782	12,557	280.3	55,766	28,488	698.0
Georgia	30,255	19,113	9,619	366.3	9,364	4,279	67.5	44,062	20,684	764.4	72,539	34,582	1,198.2
Idaho	10,696	4,674	2,999	80.3	4,136	2,756	66.0	14,512	9,218	239.6	23,322	14,973	385.9
Illinois	35,589	46,064	26,117	430.9	15,156	7,798	37.4	91,817	48,711	728.4	153,037	82,626	1,196.7
Indiana	33,117	23,955	12,945	112.6	11,241	5,753	66.4	42,772	22,615	137.1	77,968	41,313	316.1
Iowa	22,000	14,990	8,338	575.7	5,854	3,090	96.0	28,587	15,675	1,079.3	49,431	27,103	1,751.0
Kansas	21,423	9,826	4,922	768.4	6,017	3,222	99.2	23,662	11,877	1,002.8	39,505	20,021	1,870.4
Kentucky	18,442	12,429	6,732	102.3	2,961	1,763	32.7	40,354	20,504	646.2	55,744	28,999	781.2
Louisiana	18,402	9,962	4,981	74.9	6,346	3,173	91.5	42,700	20,226	461.8	59,008	28,380	628.2
Maine	8,689	8,821	4,707	71.1	776	390	5.6	13,279	6,805	116.9	22,876	11,902	193.6
Maryland	14,602	21,011	11,036	83.9	4,944	2,274	15.6	12,540	6,668	81.6	38,495	19,978	181.1
Massachusetts	27,549	9,950	4,965	25.9	3,686	1,831	6.6	43,917	20,442	41.3	57,553	27,238	73.8
Michigan	31,143	46,415	24,115	729.3	14,808	7,724	102.7	46,720	23,468	434.5	107,943	55,307	1,266.5
Minnesota	23,102	9,120	4,690	441.1	9,767	5,330	409.0	36,324	18,930	1,232.5	55,211	28,950	2,082.6
Mississippi	16,533	8,238	4,030	319.0	6,772	3,778	110.2	30,240	15,380	775.0	45,250	23,188	1,204.2
Missouri	27,096	23,103	12,257	1,035.0	3,508	1,887	5.5	79,270	41,175	1,324.1	105,881	55,319	2,364.6
Montana	21,295	9,909	6,179	210.0	6,157	3,560	138.8	23,408	14,817	419.0	39,474	24,556	767.8
Nebraska	19,146	20,499	10,657	951.1	6,025	2,731	64.9	32,694	17,958	856.9	59,218	31,346	1,872.9
Nevada	15,719	2,911	2,473	78.3	781	650	21.0	7,951	6,666	149.2	11,643	9,789	248.5
New Hampshire	8,003	1,940	970	12.7	997	498	6.1	6,572	3,374	41.2	9,509	4,842	60.0
New Jersey	25,845	14,086	6,586	57.7	8,097	4,026	17.4	28,648	13,258	59.3	50,831	23,870	134.4
New Mexico	13,876	3,328	2,106	57.8	3,978	2,583	78.2	9,545	6,105	199.2	16,851	10,794	335.2
New York	82,983	19,697	10,280	46.6	34,745	16,803	78.7	226,876	105,958	317.3	281,318	133,041	442.6
North Carolina	26,501	18,281	9,040	305.4	5,370	2,624	84.7	43,334	21,419	651.9	66,985	33,083	1,042.0
North Dakota	13,132	5,494	2,784	888.8	7,078	3,548	699.0	12,112	6,088	609.8	24,684	12,420	2,197.6
Ohio	43,949	52,607	26,404	163.6	10,889	5,350	23.4	60,735	29,005	107.2	124,231	60,759	294.2
Oklahoma	25,849	12,720	6,843	274.3	14,405	7,333	166.9	27,694	14,518	387.6	54,819	28,694	828.8
Oregon	13,611	9,127	5,458	113.7	1,934	1,156	6.6	16,741	10,341	237.1	27,802	16,955	357.4
Pennsylvania	46,777	49,629	24,567	77.1	25,981	13,307	63.9	93,190	46,331	254.8	168,800	84,205	395.8
Rhode Island	7,392	3,744	1,872	10.7	3,161	1,580	2.4	11,458	5,742	36.9	18,363	9,194	50.0
South Carolina	16,672	12,448	6,673	360.2	5,005	2,686	23.3	18,563	9,801	344.7	36,016	19,160	728.2
South Dakota	12,947	13,102	7,546	641.7	3,527	2,055	138.6	9,892	5,716	477.8	26,521	15,317	1,258.1
Tennessee	28,805	15,188	7,552	260.9	6,650	3,325	64.4	36,456	16,512	442.3	58,294	27,389	767.6
Texas	58,289	19,979	10,388	423.8	22,238	12,109	215.7	81,606	43,118	1,219.9	123,823	65,615	1,859.4
Utah	9,428	4,759	3,574	79.0	2,258	1,645	12.5	12,208	9,265	234.2	19,225	14,484	325.7
Vermont	6,504	1,820	914	17.9	103	53	.1	8,761	4,584	83.8	10,684	5,551	101.8
Virginia	25,501	12,940	6,652	220.6	5,617	2,631	67.7	23,732	11,725	299.7	42,289	21,008	588.0
Washington	16,499	17,441	9,522	175.2	3,262	1,937	62.1	21,708	11,531	137.8	42,411	22,990	375.1
West Virginia	18,853	10,067	5,140	46.8	3,824	1,905	28.0	17,579	8,840	78.9	31,470	15,885	153.7
Wisconsin	26,279	10,801	5,258	174.6	5,374	2,677	29.0	44,851	22,463	586.9	61,026	30,398	790.5
Wyoming	8,979	3,825	2,465	66.0	991	640	35.3	15,536	10,037	322.3	20,352	13,142	423.6
Hawaii	6,070	1,355	678	2.7	3,970	1,960	4.2	4,247	1,851	10.2	9,572	4,489	17.1
District of Columbia	10,498	4,977	2,489	6.6	317	158	.6	12,198	5,755	3.5	17,492	8,402	10.7
Puerto Rico	13,226	2,633	1,272	15.3	1,790	836	3.4	16,074	7,311	59.1	20,497	9,419	77.8
TOTAL	1,114,567	723,347	383,435	12,367.1	332,551	173,408	3,635.1	1,787,098	911,724	19,963.3	2,842,996	1,468,567	35,965.5

1/ Includes 1957 apportionment effective August 9, 1955.

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Travel on main rural roads increased 24 percent from 1950 to 1954

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C. M. Billingsley, *Editor*

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Washington 25, D. C.

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The printing of this publication has been approved by the Director of the Bureau of the Budget, March 17, 1955.

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

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Traffic and Travel Trends, 1954

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by THOMAS B. DIMMICK
Head, Current Data Analysis Unit

In keeping with the prime significance of highway transportation, this article discusses the growth in traffic volumes carried on rural roads and city streets by vehicle types, and presents more detailed data than were available heretofore. Material received from the several State highway departments in connection with the major highway studies of 1954 has made it possible to establish new benchmarks from which more accurate estimates of future travel and traffic trends can be based.

A comparison of rural and urban travel in 1954 shows the following highway usage: rural, 56.6 percent; and urban, 43.4 percent. In 1948, the division was about equal. Since 1949 truck travel in rural areas has increased at a faster rate than in urban areas, and passenger cars have exceeded that rate. On the other hand, truck travel in urban areas gained more rapidly than did that of passenger cars. Travel by all types of vehicles on all roads and streets increased more than 22 percent since 1950.

The growth of travel on all roads and streets by trucks and truck combinations for the period 1940–54 far exceeds that for passenger cars and buses. Truck travel increased 112 percent; passenger cars, 81 percent; and buses, 65 percent.

With the exception of the war years and a few years thereafter, the trend of total travel follows closely the economic trend as represented by the Gross National Product. Preliminary estimates of travel for 1955 indicate an increase of 4.6 percent over the previous year.

Average daily travel on all rural roads in 1954 was 869 million vehicle-miles compared with 845 million in 1953, a 2.9 percent increase. Travel on main rural roads in 1954 was 98 percent above 1940; passenger-car mileage was up 93 percent; while truck and truck-combination travel increased 116 percent.

The weights of single-unit trucks, loaded or empty, have increased only about 10 percent from 1936 to 1954. Truck-combination weights increased rather steadily throughout the period—loaded weights being some 61 percent greater and empty weights about 73 percent greater in 1954 than in 1936. Ton-mileage hauled in single-unit trucks increased from 14.3 billion in 1936 to 36.6 billion in 1954. For truck-combinations, ton-mileage increased from 13.7 billion in 1936 to 107.3 billion in 1954. In 1950, 18.5 percent of the truck and truck-combination travel was by two-axle tractors pulling single-axle semitrailers, while travel by the same type of tractor with dual-axle semitrailers amounted to only 9 percent. By 1954 this relation had changed materially, the dual-axle semitrailers being considerably more numerous. In 1954, the four-axle combinations also accounted for a larger portion of the ton-mileage than any other single vehicle type, whereas in 1950 the three-axle combinations predominated.

This change is reflected in axle loads. The frequency of axles weighing 18,000 pounds or more in 1950 was over seven times that in 1936, but from the 1950 high there was a consistent yearly drop which amounted to 35 percent by 1954. The frequencies for 20,000- and 22,000-pound axles decreased more than one-half during the same period. While the frequency of loads exceeding State legal limits by 5 percent changed little, there appears to be a real reduction in the percentage of higher overloads. Overloads of 50 percent were rare after 1951, and overloads of 20 to 50 percent declined noticeably.

THE ever-increasing importance of our highways in the movement of persons from point to point and of goods from producer to consumer makes a thorough knowledge of such movements, their volumes, present trends, and potential future levels of great interest and importance to persons engaged in transportation studies or in the design of traffic facilities.

In the 5-year period beginning in 1936 (mostly in 1936 and 1937), 47 of the 48 States in cooperation with the Bureau of Public Roads conducted surveys for a 12-month period to collect data which would supply comprehensive information concerning vehicle characteristics and travel habits. The measuring of road mileages, the counting of traffic by vehicle type, the weighing of trucks on rural roads, and the questioning of drivers concerning origin and destination and miles driven on different road systems during the preceding year supplied basic data from which a vast amount of information regarding travel habits, ton-miles hauled on rural systems, and vehicle-miles driven on all systems could be calculated for the period of the survey.

Since the original surveys, the States have operated automatic traffic recorders at a large number of locations and have adopted other continuing operations which provide sufficient data for estimating trends in traffic volumes. Periodic weighing operations, combined with manual classified counts of all vehicles passing the weighing stations, have been made which provide information concerning vehicle types and weights as well as their loading habits. By means of these trends, annual estimates were published showing for each year the travel on rural roads from 1936 to 1952 and on city streets from 1936 to 1948.[1] By combining carried load data with vehicle-mileage figures, the ton-mileage of freight hauled on main rural roads was estimated for each year. Sufficient data are not available to justify publishing estimates of ton-mileage carried on local roads. No data are available concerning loads carried on city streets and no attempt has been made to estimate the amount of this haulage.

Between 1946 and 1950, estimated volumes of total urban and rural travel from 1936 to 1948, inclusive, were published annually in PUBLIC ROADS. These estimates were based on comprehensive statewide traffic surveys made in the 1936–37 period, and were adjusted each year by the use of variable indices such as motor-fuel consumption and motor-vehicle registration. Following the issuance of the 1948 data, it became apparent that significant changes in factors affecting travel had occurred. The decision was made at that time to discontinue publishing the series until the States had collected a sufficient body of current data on urban travel.

Urban Travel Data Improve

Many States now have adopted some plan of observing the trends in urban travel either by means of continuous or part-time counts, and several have made sufficient counts to enable them to estimate accurately vehicle-miles of travel on the streets of their key cities. Most States are now able to report the average daily travel on the urban extensions of State highway systems and other arterial streets of their cities, and to estimate with a reasonable degree of accuracy the travel on all of the streets.

Although approximations may have to be made concerning the lightly traveled local streets, a large expenditure of funds to determine accurately the amount of travel on them is not justifiable. Motor-vehicle-use surveys, made in several States in recent years, have

[1] See previous articles on traffic in PUBLIC ROADS: vol. 27, Nos. 6 and 11; vol. 26, Nos. 5 and 11; vol. 25, Nos. 3, 7, and 12; vol. 24, No. 10; and vol. 23, No. 9.

364144—55——1

Table 1.—Estimate of motor-vehicle travel in the United States by vehicle types, 1949–54

Vehicle type	Travel in millions of vehicle-miles for calendar years—					
	1949	1950	1951	1952	1953	1954
Passenger cars: [1]						
Rural travel	168,897	181,095	205,375	222,064	237,567	246,300
Urban travel	173,581	182,518	186,756	188,123	197,784	204,305
Total	342,478	363,613	392,131	410,187	435,351	450,605
Commercial buses:						
Rural travel	1,470	1,394	1,381	1,444	1,455	1,453
Urban travel	2,030	1,877	1,822	1,750	1,856	1,743
Total	3,500	3,271	3,203	3,194	3,311	3,196
School and nonrevenue buses:						
Rural travel	677	729	823	1,026	1,024	1,091
Urban travel	75	81	92	114	114	121
Total	752	810	915	1,140	1,138	1,212
All buses:						
Rural travel	2,147	2,123	2,204	2,470	2,479	2,544
Urban travel	2,105	1,958	1,914	1,864	1,970	1,864
Total	4,252	4,081	4,118	4,334	4,449	4,408
All passenger vehicles:						
Rural travel	171,044	183,218	207,579	224,534	240,046	248,844
Urban travel	175,686	184,476	188,670	189,987	199,754	206,169
Total	346,730	367,694	396,249	414,521	439,800	455,013
Trucks and combinations:						
Rural travel	48,053	56,780	60,843	64,929	68,329	68,374
Urban travel	29,678	33,772	34,001	34,131	36,304	37,470
Total	77,731	90,552	94,844	99,060	104,633	105,844
All motor vehicles:						
Rural travel	219,097	239,998	268,422	289,463	308,375	317,218
Urban travel	205,364	218,248	222,671	224,118	236,058	243,639
Total	424,461	458,246	491,093	513,581	544,433	560,857

[1] Includes taxicabs.

provided additional data concerning travel on all types of city streets. While much is desired in the way of more complete urban travel data for the major thoroughfares, considerable progress is being made in the collection of this information.

New Travel Trend Base Established

The Federal-Aid Highway Act of 1954 directed that a study be made of the costs of completing the several systems of highways in the several States. One important result of the States' cooperation in this study was a complete estimate of vehicle-miles of travel in 1953 on all systems, urban as well as rural. These State estimates, based on the best available data regarding current travel on the various systems of roads and streets, now make it possible to establish a new base for national vehicle-mileage estimates.

The new vehicle-mile totals check closely with those previously calculated from trend data since 1948. The new main rural road figure is only a small fraction of one percent larger than the old trend figure. Somewhat larger discrepancies were found between the old and new local rural road and urban street estimates although the totals are practically the same. In fact, the new total vehicle-mileage estimate exceeded the old trend total by only 0.4 of one percent. Such close concurrence of the two estimates indicates that the plan of adjusting vehicle-mileage figures by means of trends is satisfactory for the main highway systems, provided the estimates are checked periodically to adjust for the mileage changes that occur.

Urban Travel Estimates Extended

Since the 1953 vehicle-mileage total compared so favorably with the figure derived from the State reports, all estimates for the years 1949–54 have been adjusted to remove the discrepancy indicated by the 1953 check. Table 1 gives the estimated travel in millions of vehicle-miles by passenger cars, commercial buses, school and nonrevenue buses, and trucks on rural roads and on urban streets for the 6 years commencing with 1949 when publication of urban travel estimates was discontinued. A comparison of the figures indicates that truck travel is increasing at a faster rate in rural areas than in urban. At the same time, travel by passenger cars is increasing even faster on rural roads than truck travel. Conversely, truck travel on urban streets is increasing faster than passenger-car travel. Commercial-bus travel, both rural and urban, is decreasing while school bus travel is rapidly increasing. Total travel has increased over 22 percent in the 4 years since 1950.

Travel Follows Economic Trend

Travel on all rural roads and streets, motor-vehicle registration, motor-fuel consumption, and the Gross National Product are shown in figure 1 for the years 1936–54, inclusive, as a percentage of the 1950 values. This chart indicates that, with the exception of the war years and a few years thereafter when traffic restrictions drastically curtailed travel while production was stimulated, the trend of total travel follows closely the economic trend as represented by the Gross National Product.

Immediately following the war, the Gross National Product declined and then leveled off while traffic increased rapidly. The curves came together again in 1949 and followed closely until 1954. The downward trend of production in 1954 (which proved to be only temporary) was reflected only slightly in highway travel; the rate of increase was 3 percent from 1953 to 1954 compared with 6 percent from 1952 to 1953. The volume of truck travel in 1954, however, was only 1.2 percent above the previous year's total and ton-mileage hauled in 1954 was 2 percent less than in the previous year. Preliminary estimates of travel and Gross National Product for 1955 indicate that both quantities will be about 4.6 percent greater than in the previous year.

Figure 2 shows traffic volumes on rural roads and urban streets for each of the years 1936–54, as a percentage of such travel in 1950. It is noticeable that from 1936 to 1948 the curves representing rural roads and urban streets are very similar. From 1948 on, however, rural travel increased at a fairly uniform rate with only a slight indication of leveling off, while urban travel increased each year but at a much reduced rate, which resulted in a considerable spread between the two curves. The past year was an exception to the general

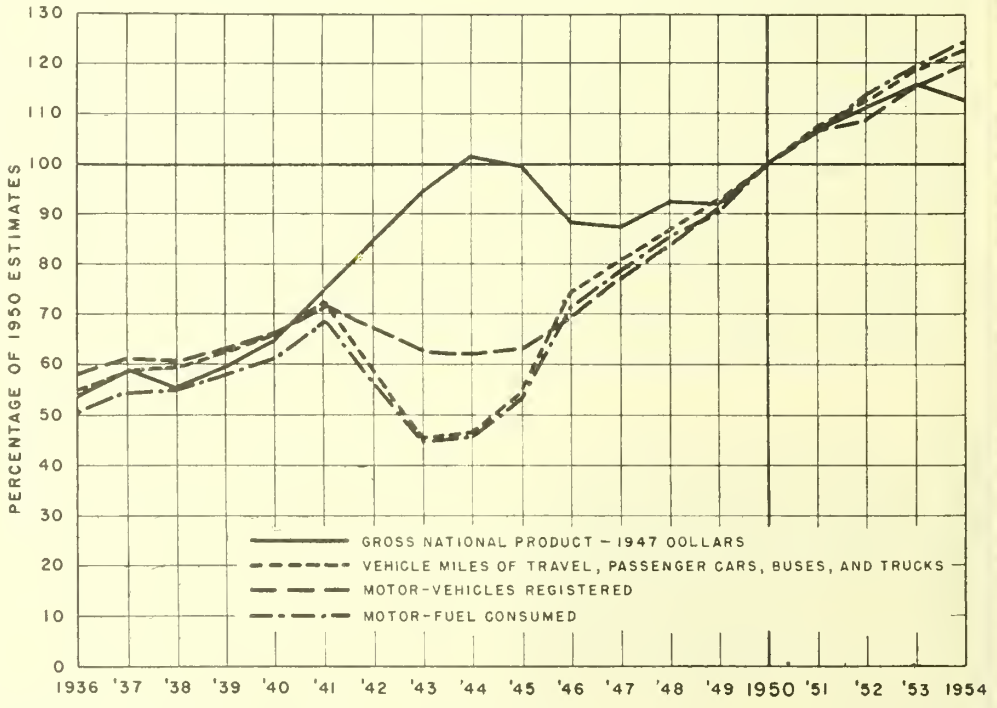

Figure 1.—Total travel, motor-vehicle registration, motor-fuel consumption, and Gross National Product, 1936–54, as a percentage of the respective amounts in 1950.

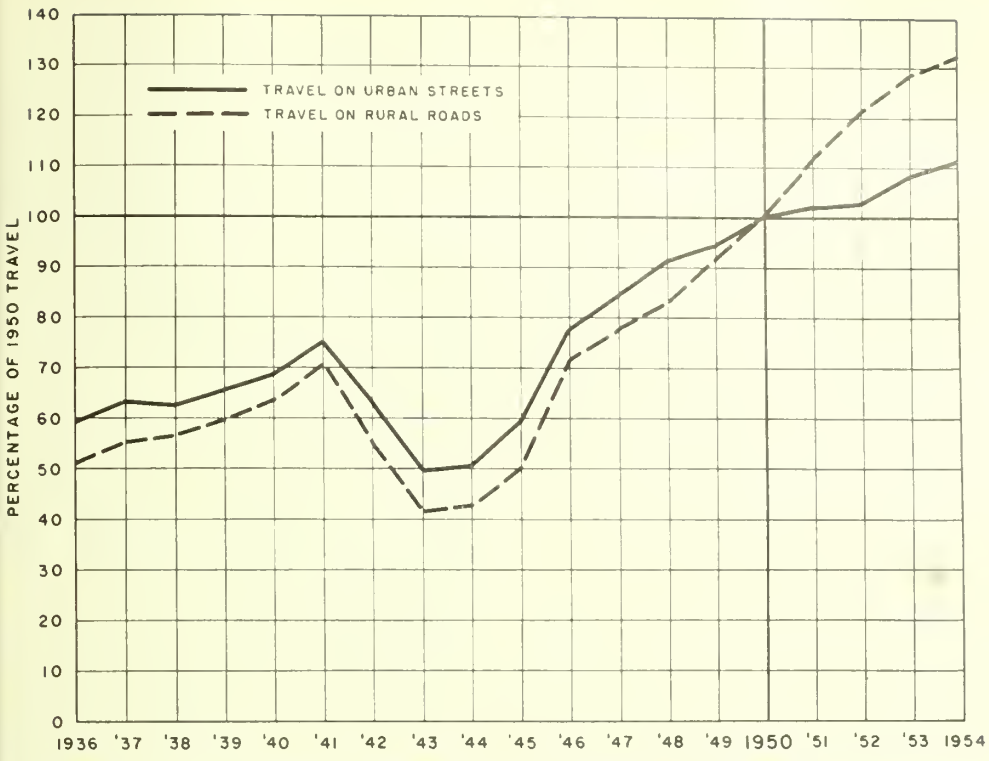

Figure 2.—Travel on all rural roads and streets, 1936–54, as a percentage of such travel in 1950.

trend that prevailed from 1948 through 1953, since in 1954 urban travel increased 3.2 percent while rural travel increased only 2.9 percent. This difference is small and probably of little significance.

In 1948 and previous years for which estimates were made, the total volumes of urban and rural travel were approximately equal. For instance, the 1936 estimates indicated that rural traffic accounted for 48.66 percent of all travel, while in 1948 the figure was 49.97 percent. Since 1948, rural travel has exceeded urban travel to an increasing degree. The relation of rural travel to total travel in the succeeding years was as follows: 51.62 percent, 1949; 52.37 percent, 1950; 54.66 percent, 1951; 56.36 percent, 1952; 56.64 percent, 1953; and 56.56 percent, 1954.

Table 2 shows the estimated amounts of travel in 1954 on main rural roads, local rural roads, and urban streets for passenger cars, buses, and trucks together with the number of vehicles registered and the quantity of motor fuel consumed. The travel figures were obtained by applying the available trends to the 1953 data, which, as previously stated, were derived from the various State reports submitted for the nationwide highway study.

In general, the table is comparable to the one compiled for a study made in 1948.[2] In addition to traffic volumes and data concerning vehicles registered and motor fuel consumed, table 2 shows average miles of travel per vehicle, average consumption of motor fuel per vehicle, and average travel per gallon of fuel consumed. One feature to be found in this table, and not included in the 1948 study, is the segregation of travel data on main roads and local roads.

[2] *Trends in motor-vehicle travel, 1948*, by G. P. St. Clair. PUBLIC ROADS, vol. 25, No. 12, Feb. 1950, p. 296.

A comparison of vehicle-miles of travel on all roads and streets in 1940, 1945, 1950, 1953, and 1954 is given in table 3. Probably the most significant relation shown by this table is the greater increase of travel by trucks and truck combinations in comparison with passenger cars and buses until 1950. Travel by trucks and truck combinations in 1954 was 112 percent greater than in 1940. The total increase for passenger-car travel during the period was 81 percent. Bus travel leveled off considerably and only a 65-percent gain is indicated for this period. The percentage of all rural and urban travel by trucks and combinations increased from 16.52 percent in 1940 to 18.87 percent in 1954.

Rural-Road Travel Increases

Figure 3 shows the annual vehicle-miles of travel on all rural roads by 12-month periods ending each month (moving average) from the end of 1936, the first year of the planning surveys, to the present. This method of presentation reduces the seasonal fluctuations. The portion of the curve from the end of 1946 through 1950 indicates that the increases averaged over 10 percent each year and was almost 12 percent during 1951. During 1952 and 1953, the annual increases were about 7 percent and in 1954, the increase dropped to 3 percent. Reports for the first half of 1955 indicate an increase of 4 to 5 percent for the calendar year.

Travel on all rural roads in the eastern, central, and western regions of the country,[3] by 12-month periods ending each month, is shown in figure 4 as a percentage of such travel in 1950. Since 1950, traffic has consistently increased more in the eastern regions than in other sections of the country which is the reverse of what occurred from 1941 to 1950.[4] In the western regions, traffic increased slowly in 1951 but sharply reduced the eastern lead in 1952 and 1953, while in 1954 the trend was similar to that in the eastern regions and only slightly lower. Travel in the central regions followed a trend below those of either the eastern or western regions, but made an

[3] *Eastern regions.*—New England: Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont. Middle Atlantic: New Jersey, New York, and Pennsylvania. South Atlantic: Delaware, Florida, Georgia, Maryland, North Carolina, South Carolina, Virginia, and West Virginia. *Central regions.*—East North Central: Illinois, Indiana, Michigan, Ohio, and Wisconsin. East South Central: Alabama, Kentucky, Mississippi, and Tennessee. West North Central: Iowa, Kansas, Minnesota, Missouri, Nebraska, North Dakota, and South Dakota. West South Central: Arkansas, Louisiana, Oklahoma, and Texas. *Western regions.*—Mountain: Arizona, Colorado, Idaho, Montana, Nevada, New Mexico, Utah, and Wyoming. Pacific: California, Oregon, and Washington.

[4] *Trends in traffic volumes, vehicle types, and weights*, by Thomas B. Dimmick. PUBLIC ROADS, vol. 27, No. 11, Dec. 1953, p. 236.

Table 2.—Estimate of motor-vehicle travel in the United States by vehicle types in calendar year 1954

Vehicle type	Motor-vehicle travel					Number of registered vehicles [1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total [2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles per gal.*
Passenger cars [3]	169,755	76,545	246,300	204,305	450,605	48,413	9,308	30,915	639	14.58
Buses:										
Commercial	1,148	305	1,453	1,743	3,196	83	38,506	639	7,699	5.00
School and nonrevenue	603	488	1,091	121	1,212	150	8,080	116	773	10.45
All buses	1,751	793	2,544	1,864	4,408	233	18,918	755	3,240	5.84
All passenger vehicles	171,506	77,338	248,844	206,169	455,013	48,646	9,354	31,670	651	14.37
Trucks and combinations	45,553	22,821	68,374	37,470	105,844	9,726	10,883	12,541	1,289	8.44
All motor vehicles	217,059	100,159	317,218	243,639	560,857	58,372	9,608	44,211	757	12.69

[1] Registration figures differ slightly from those in Bureau of Public Roads table MV-1 for 1954 because of adjustments for defective classification in a few States and to allow for duplicate registrations.
[2] Total fuel consumed differs from that given in Bureau of Public Roads table G-21 because of adjustments to cover estimated amounts used by motorcycles.
[3] Includes taxicabs.

Table 3.—Comparison of the estimated vehicle-miles of travel on all roads and streets in 1940, 1945, 1950, 1953, and 1954

Year	All vehicles, vehicle-miles	Passenger cars		Buses		Trucks and combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*
1940	302,188	82.60	249,604	0.88	2,657	16.52	49,927
1945	250,173	80.02	200,199	1.53	3,832	81.45	46,142
1945: 1940 ratio	*.83*	*.97*	*.80*	*1.74*	*1.44*	*1.12*	*.92*
1950	458,246	79.35	363,613	.89	4,081	19.76	90,552
1950: 1945 ratio	*1.83*	*.99*	*1.82*	*.58*	*1.06*	*1.07*	*1.96*
1950: 1940 ratio	*1.52*	*.96*	*1.46*	*1.01*	*1.54*	*1.20*	*1.81*
1953	544,433	79.96	435,351	.82	4,449	19.22	104,633
1953: 1950 ratio	*1.19*	*1.01*	*1.20*	*.92*	*1.09*	*.97*	*1.16*
1953: 1945 ratio	*2.18*	*1.00*	*2.17*	*.54*	*1.16*	*1.04*	*2.27*
1953: 1940 ratio	*1.80*	*.97*	*1.74*	*.93*	*1.67*	*1.16*	*2.10*
1954	560,857	80.34	450,605	.79	4,408	18.87	105,844
1954: 1953 ratio	*1.03*	*1.00*	*1.04*	*.96*	*.99*	*.98*	*1.01*
1954: 1950 ratio	*1.22*	*1.01*	*1.24*	*.89*	*1.08*	*.95*	*1.17*
1954: 1945 ratio	*2.24*	*1.00*	*2.25*	*.52*	*1.15*	*1.02*	*2.27*
1954: 1940 ratio	*1.85*	*.97*	*1.81*	*.90*	*1.65*	*1.14*	*2.12*

average annual gain over the period of about 6.5 percent.

The average daily vehicle-miles of travel on all rural roads by months in 1953, 1954, and the first 8 months of 1955 are shown graphically in figure 5. The graph shows that travel in 1954 was generally heavier month by month in all regions than it was in the previous year, except in the western regions during June. Likewise for the first 8 months of the year, travel in 1955 exceeded that in 1954 each month except in the central regions in February and in the western regions in April.

The average daily travel on all rural roads in the United States in 1953 was 845 million vehicle-miles. The 1954 figure of 869 million vehicle-miles exceeded 1953 by 2.9 percent. Daily travel in the eastern regions in 1953 amounted to 278 million vehicle-miles. In 1954, travel in these regions increased 3.2 percent over the previous year and amounted to 287 million vehicle-miles. In the central regions, the average daily travel in 1953 was 414 million vehicle-miles. This figure was increased 2.2 percent in 1954 and amounted to 423 million vehicle-miles. In the western regions, travel in 1953 amounted to 153 million vehicle-miles daily, and increased 3.9 percent in 1954, or to 159 million vehicle-miles.

Figure 6 shows, in bar-chart form, the vehicle-miles of travel by single-unit trucks and by truck combinations on main roads and local roads. The chart illustrates the prevalence of truck combinations on main roads and, conversely, the relative infrequence of these vehicles on the local roads. Travel by truck combinations constituted almost one-third of all travel by freight-carrying vehicles on the main roads, while this means of transportation composed less than one-tenth of the total on the local roads. Put in terms of averages, one mile of main road carried 106 tractor-trailer combinations per day in 1954, while the equivalent figure for local roads was less than 3.

Average daily traffic by the lighter-weight single-unit trucks is also far less important on local than on main roads, although the difference is not as great as it is for the combinations. The average mile of main rural road carried 228 single-unit trucks daily in 1954, while only 21 such vehicles traveled the average mile of local road.

Very few data have been collected concerning the loads carried on local roads, and current traffic information concerning this mileage is not as reliable as it is for main roads. Local road mileage far exceeds that of main roads, yet estimates indicate that total truck travel on main roads was more than double and ton-mileage hauled was about four times the amount carried on local roads. Because of a scarcity of basic and trend data and the relative unimportance of the local road mileage from a freight-carrying standpoint, subsequent sections of this article will be confined to data concerning travel and freight transported on main rural roads.

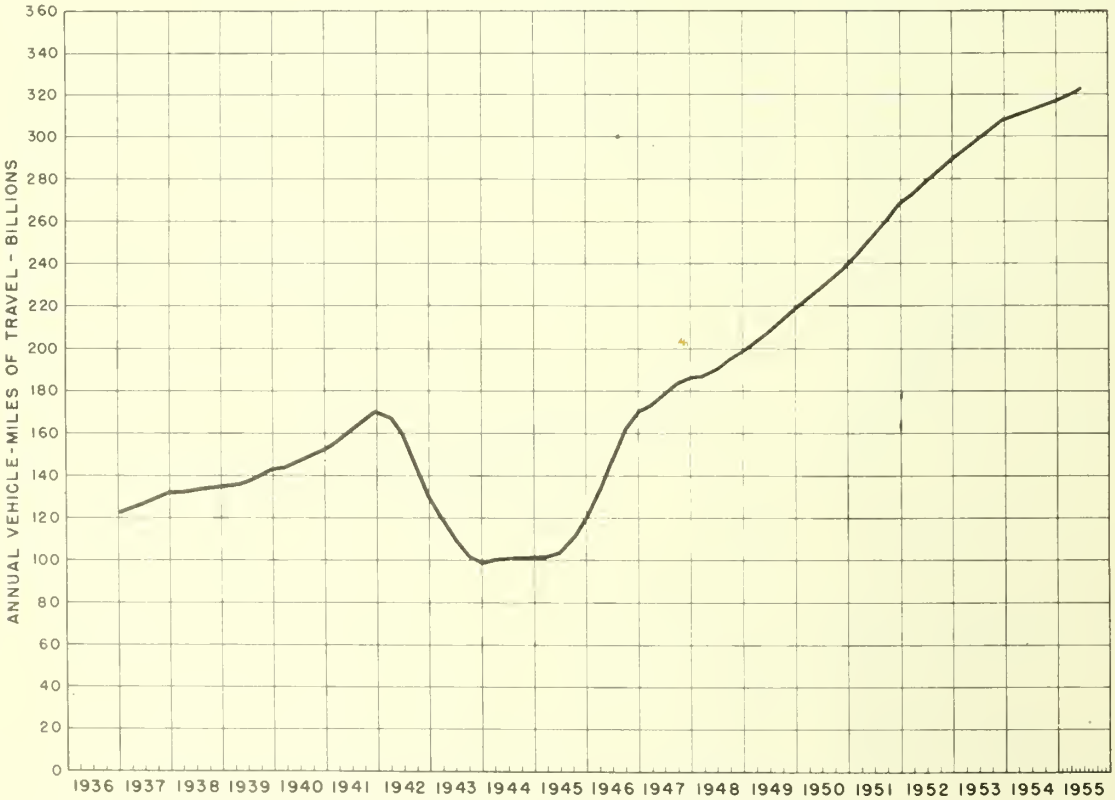

Figure 3.—Vehicle-miles of travel on all rural roads by 12-month periods ending each month, 1936 to mid-year 1955.

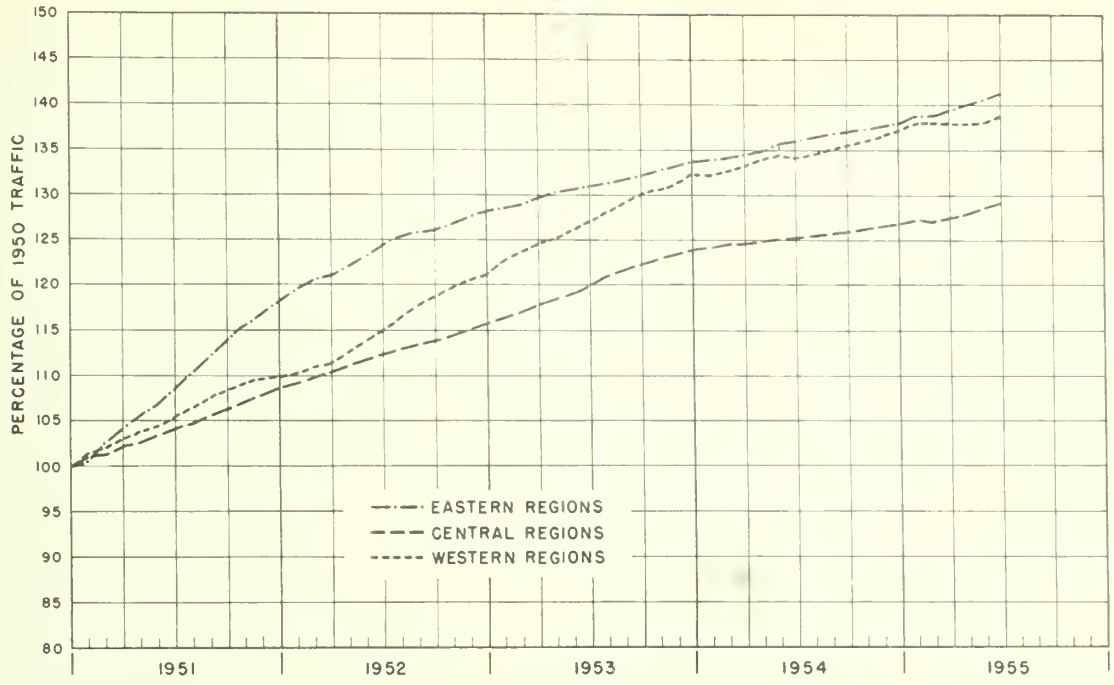

Figure 4.—Travel on all rural roads by 12-month periods ending each month, 1951 to mid-year 1955, as a percentage of travel in 1950.

Main Rural-Road Travel Continues Upward Trend

Travel by passenger vehicles, single-unit trucks, and tractor-trailer combinations on main rural roads from 1936 to 1954, inclusive, as a percentage of such travel in 1950 is shown in figure 7. Probably the most interesting and important trend is the increase in the popularity of combination-type vehicles between 1941 and 1950. From 1936 to 1941, the use of combinations was increasing at about the same rate as that of the other types of vehicles. During the war, however, the greater operating efficiency of combinations was recognized and their travel was curtailed only slightly, whereas travel by other types of trucks was reduced greatly. After the war ended, the use of combinations increased rapidly.

The 1950 surveys indicated a gain of almost 24 percent over the previous year for combination vehicles, whereas the use of single-unit trucks increased only 11 percent in the same period. Since 1950 the trends in use of combinations and of single-unit trucks have been very similar and have not climbed as fast as the trend for passenger-car use. For the entire period from 1936 to 1954, inclusive, annual travel of passenger cars increased 135 percent; single-unit trucks, 149 percent; and tractor-trailer combinations, 425 percent.

Table 4 compares the estimated vehicle-miles of travel on main rural roads in 1940, 1945, 1950, 1953, and 1954, and gives the percentage distribution of this travel by main vehicle types. The table shows that total travel in 1954 was 98 percent greater than it

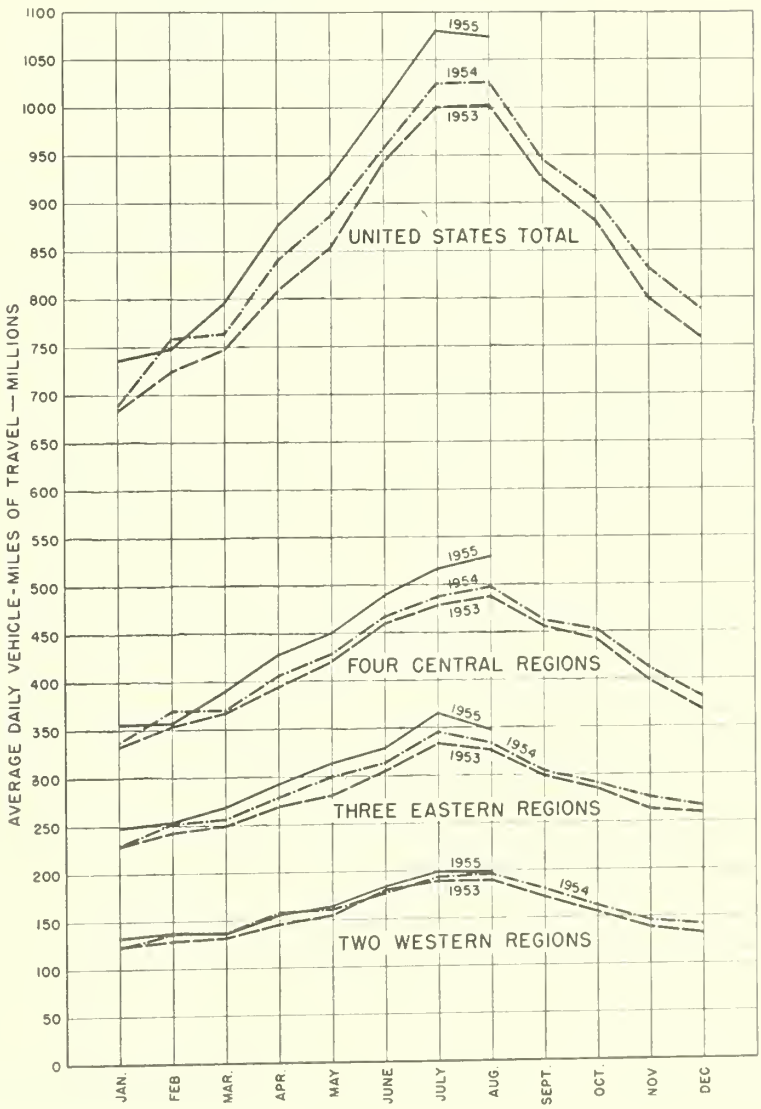

Figure 5* (Right).—*Average daily travel on all rural roads in 1953, 1954, and the first 8 months of 1955.

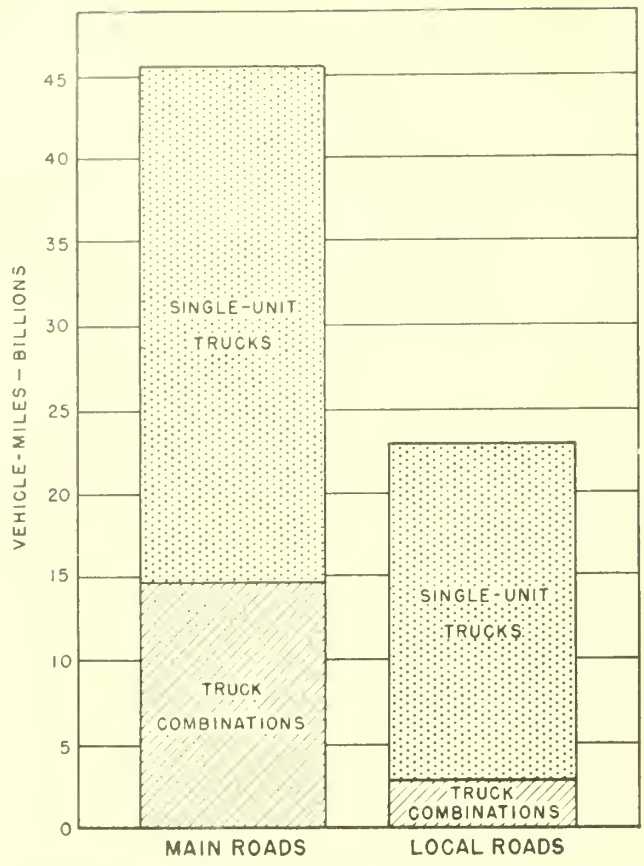

Figure 6.—Vehicle-miles of travel by trucks and truck combinations on main and local rural roads in 1954.

was in 1940, that passenger-car travel was 93 percent greater, while truck and truck-combination travel increased 116 percent. Single-unit truck travel increased 86 percent (less than passenger cars), while tractor-trailer combination travel increased 229 percent. Other ratios listed show greater increases for combinations than for single-unit trucks in each instance except in the 1954:1953 comparison. In the latter case both ratios show a slight decrease—combinations decreasing somewhat more than the smaller vehicles. This would be expected in light of the 1954 decline in Gross National Product previously mentioned.

Truck Travel Decreased in 1954

The ratios of 1954 traffic on main rural roads to corresponding traffic in 1953, by type of vehicle and by United States Census regions and the three main geographic regions, are given in table 5. For the United States as a whole, while local passenger-car travel increased about 4 percent in 1954 compared with the previous year, travel by all other types of vehicles (out-of-State passenger cars, trucks, truck combinations, and buses) decreased. Apparently the 1954 decline in Gross National Product affected tourist passenger-car travel as well as truck and bus travel, but had little effect on local passenger-car travel. The maximum gain in total travel, 3 percent, was found in the New England, South Atlantic, and West South Central regions. There was an increase in all other regions except the East South Central where there was no change, and the East North Central where there was a decrease of 1 percent.

The percentage of travel by vehicle types on main rural roads in the summer of 1954 is given in table 6 for the nine census regions and the three main geographic regions. The table shows that the largest percentage of passenger-car travel is in New England with the percentages in the Pacific and East North Central regions following closely. A comparison of truck travel shows that the East South Central region has the largest percentage of travel by all types of trucks and combinations, with the West South Central and Mountain regions following in order. The lowest percentage is in New England. Travel by all types of combinations in the East North Central region exceeds all other regions and is followed in order by the Middle Atlantic and the West North Central. When the comparison is restricted to travel by truck and full-trailer combinations, the Pacific and Mountain regions are the only ones with an appreciable proportion of vehicles of this type.

When the percentage distribution of travel by types of motor-vehicles given in table 6 is compared with a similar distribution previously published for 1950, it is found that during the 1950–54 period, the percentage of passenger-car travel increased from 76.15 to 78.28 percent; that of single-unit trucks decreased from 15.63 to 14.31 percent; and combinations decreased from 7.21 to 6.68 percent. These figures show, as was noted in the discussion concerning table 5, that travel by trucks and combinations has not expanded as fast as passenger-car travel since 1950.

Weight Stations Operated

During the summer of 1954, a total of 516 loadometer or pitscale stations were operated in 42 States for the purpose of collecting trend data concerning vehicle types, weights, and loading practices. During this survey, 395,050 trucks and truck combinations passing the stations were counted and classified according

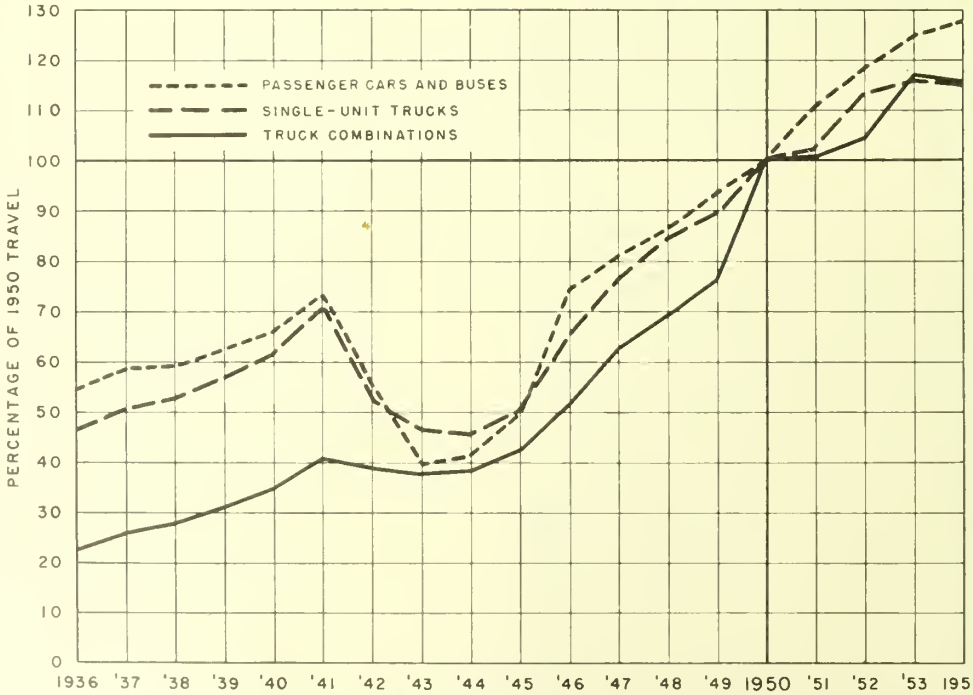

Figure 7.—Travel by passenger vehicles, single-unit trucks, and truck combinations on main rural roads, 1936–54, as a percentage of such travel in 1950.

Table 4.—Comparison of estimated vehicle-miles of travel on main rural roads in 1940, 1945, 1950, 1953, and 1954

Year	All vehicles, vehicle-miles	Passenger cars and buses		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1940	109,815	80.8	88,715	19.2	21,100	79.1	16,699	20.9	4,401
1945	85,792	78.0	66,885	22.0	18,907	71.9	13,602	28.1	5,305
1945: 1940 ratio	*.78*	*.97*	*.75*	*1.15*	*.90*	*.91*	*.81*	*1.34*	*1.21*
1950	174,349	77.2	134,527	22.8	39,822	68.4	27,257	31.6	12,565
1950: 1945 ratio	*2.03*	*.99*	*2.01*	*1.04*	*2.11*	*.95*	*2.00*	*1.12*	*2.37*
1950: 1940 ratio	*1.59*	*.96*	*1.52*	*1.19*	*1.90*	*.86*	*1.63*	*1.51*	*2.86*
1953	213,604	78.4	167,445	21.6	46,159	68.1	31,435	31.9	14,724
1953: 1950 ratio	*1.23*	*1.02*	*1.24*	*.95*	*1.16*	*.99*	*1.15*	*1.01*	*1.17*
1953: 1945 ratio	*2.49*	*1.01*	*2.50*	*.98*	*2.44*	*.95*	*2.31*	*1.14*	*2.78*
1953: 1940 ratio	*1.95*	*.97*	*1.89*	*1.13*	*2.19*	*.86*	*1.88*	*1.53*	*3.35*
1954	217,059	79.0	171,506	21.0	45,553	68.2	31,064	31.8	14,489
1954: 1953 ratio	*1.02*	*1.01*	*1.02*	*.97*	*.98*	*1.01*	*.99*	*.99*	*.98*
1954: 1950 ratio	*1.24*	*1.02*	*1.27*	*.92*	*1.14*	*.99*	*1.14*	*1.01*	*1.15*
1954: 1945 ratio	*2.53*	*1.01*	*2.56*	*.95*	*2.41*	*.95*	*2.28*	*1.13*	*2.73*
1954: 1940 ratio	*1.98*	*.98*	*1.93*	*1.09*	*2.16*	*.86*	*1.86*	*1.52*	*3.29*

to the number of axles and tire equipment. Of that number, 134,488 vehicles were weighed and a record was made of the type of each vehicle, the weight of each of its axles, the spacing in feet between each pair of axles, and whether the vehicle was loaded or empty.

The stations used in the 1954 survey were located at the same points as in former years, most of them being at sites operated in the original surveys in the 1936–40 period. From comparable data collected at these locations trends in travel, loading practices, and carried loads were obtained, which, when applied to former estimates derived from comprehensive surveys, gave current estimates of vehicle-miles traveled by loaded vehicles and the carried load. The product of these two factors is the ton-miles of carried load. Data concerning the frequency of overloading and of heavy axle and heavy gross weight occurrence also are made available. The remaining

Table 5.—Ratio of 1954 traffic on main rural roads to corresponding traffic in 1953, by regions

Vehicle type	Eastern regions [1]				Central regions [2]					Western regions [3]			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.04	1.04	1.07	1.06	1.01	1.00	1.06	1.02	1.02	1.04	1.04	1.04	1.04
Foreign	1.02	.96	1.01	1.00	.99	.97	.97	1.09	1.00	.96	.99	.97	.99
All passenger cars	1.03	1.03	1.05	1.04	1.00	.99	1.04	1.03	1.01	1.00	1.03	1.02	1.03
Trucks and truck combinations:													
Single-unit trucks	1.03	.99	.94	.97	.95	1.06	.92	1.01	.99	1.07	.99	1.03	.99
Truck combinations	.96	1.02	.94	.97	.92	.91	1.04	1.14	.99	1.04	.98	1.10	.98
All trucks and combinations	1.01	1.00	.94	.97	.93	1.03	.96	1.04	.99	1.06	.99	1.02	.99
Buses	1.04	.95	.94	.96	.86	.90	.81	.83	.85	1.02	.91	.95	.91
All vehicles	1.03	1.02	1.03	1.03	.99	1.00	1.02	1.03	1.00	1.02	1.02	1.02	1.02

[1] Includes toll road vehicle-mileage for Maine, New Hampshire, New Jersey, and Pennsylvania.
[2] Includes toll road vehicle-mileage for Oklahoma (Turner Turnpike).
[3] Includes toll road vehicle-mileage for Colorado (Denver-Boulder Turnpike).

Table 6.—Percentage distribution of travel by vehicle types on main rural roads in the summer of 1954, by regions

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	62.22	64.89	58.64	61.32	61.85	49.73	62.91	59.84	59.62	39.45	69.48	58.19	59.93
Foreign	21.58	14.40	20.28	18.45	18.80	21.65	17.01	13.64	17.55	35.81	11.27	20.50	18.35
All passenger cars	83.80	79.29	78.92	79.77	80.65	71.38	79.92	73.48	77.17	75.26	80.75	78.69	78.28
Single-unit trucks:													
Panel and pickup	3.70	3.66	7.15	5.44	5.21	11.42	5.84	11.46	7.94	10.99	7.03	8.52	7.22
Other 2-axle, 4-tire	1.47	1.43	.48	.95	.23	.12	.46	.25	.27	1.02	1.10	1.07	.63
Other 2-axle, 6-tire	5.58	6.80	5.72	6.07	4.76	9.65	5.89	7.04	6.37	5.48	3.78	4.42	5.93
3-axle	.50	.47	.77	.63	.40	.67	.43	.17	.39	.68	.73	.71	.53
All single-unit trucks	11.25	12.36	14.12	13.09	10.60	21.86	12.62	18.92	14.97	18.17	12.64	14.72	14.31
Truck-tractor and semitrailer combinations:													
3-axle	3.18	5.20	2.43	3.50	3.16	3.41	2.22	2.95	2.94	1.22	.67	.88	2.77
4-axle	.83	2.34	3.53	2.72	4.08	2.25	3.59	3.61	3.56	1.09	.76	.88	2.82
5-axle or more	.01	.03	.02	.02	.51	.04	.84	.22	.43	2.43	2.71	2.60	.68
All truck-tractor and semitrailer combinations	4.02	7.57	5.98	6.24	7.75	5.70	6.65	6.78	6.93	4.74	4.14	4.36	6.27
Truck and trailer combinations:													
4-axle or less	.01	.04	.04	.03	.07	.01	.25	.13	.12	.30	.21	.24	.11
5-axle	------	.03	------	.01	.32	------	.03	.01	.13	.72	.86	.81	.21
6-axle or more	------	------	------	------	.10	------	(1)	------	.03	.15	.62	.45	.09
All truck and trailer combinations	.01	.07	.04	.04	.49	.01	.28	.14	.28	1.17	1.69	1.50	.41
All combinations	4.03	7.64	6.02	6.28	8.24	5.71	6.93	6.92	7.21	5.91	5.83	5.86	6.68
All trucks and truck combinations	15.28	20.00	20.14	19.37	18.84	27.57	19.55	25.84	22.18	24.08	18.47	20.58	20.99
Buses	.92	.71	.94	.86	.51	1.05	.53	.68	.65	.66	.78	.73	.73
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

[1] Less than 0.005 percent.

Table 7.—Average weight (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1954, by regions

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Average Weights of Loaded Vehicles													
Single-unit trucks:													
Panel and pickup	5,415	4,918	5,430	5,302	5,177	5,509	5,339	5,566	5,405	5,316	4,924	5,083	5,307
Other 2-axle, 4-tire	6,121	6,100	7,109	6,370	6,567	6,763	7,464	6,754	6,993	7,076	6,371	6,622	6,582
Other 2-axle, 6-tire	15,797	14,831	14,497	14,806	13,204	15,277	14,332	13,565	14,016	14,721	14,273	14,461	14,337
3-axle	40,396	34,220	30,441	32,490	29,788	28,790	27,870	30,470	29,129	31,657	30,003	30,420	30,702
Average	12,884	12,523	12,184	12,401	10,594	11,431	10,874	9,607	10,583	10,091	10,228	10,173	11,021
Truck combinations:													
Truck-tractor and semitrailer	40,593	38,010	41,415	39,934	41,245	38,262	42,726	40,090	40,926	50,845	54,445	53,077	42,327
Truck and trailer		49,620	54,200	52,414	68,524	46,228	34,141	39,968	53,451	64,336	62,226	62,764	59,422
Average	40,593	38,057	41,488	39,991	42,382	38,266	42,324	40,087	41,350	52,872	56,315	55,096	43,259
Average, all trucks and combinations	21,348	24,046	23,700	23,531	26,251	18,270	23,572	20,680	22,796	24,344	27,600	26,354	23,661
Average Weights of Empty Vehicles													
Single-unit trucks:													
Panel and pickup	4,495	3,784	4,032	4,013	3,958	4,159	4,350	4,148	4,133	4,029	4,156	4,085	4,091
Other 2-axle, 4-tire	4,569	4,454	5,597	4,760	4,796	6,093	6,165	5,435	5,507	5,269	4,821	4,983	4,976
Other 2-axle, 6-tire	10,112	8,682	8,094	8,591	7,949	8,271	8,206	8,147	8,149	8,656	8,410	8,546	8,340
3-axle	22,231	15,420	13,535	15,138	14,416	12,465	14,302	14,173	13,758	17,161	16,089	16,691	14,912
Average	7,658	6,576	5,680	6,182	5,652	6,006	6,280	5,470	5,768	5,624	5,530	5,580	5,865
Truck combinations:													
Truck-tractor and semitrailer	23,172	20,258	21,052	20,917	21,353	19,691	21,629	20,259	20,846	26,721	27,064	26,883	21,325
Truck and trailer	19,650	23,642	27,450	24,782	27,544	23,416	17,092	21,284	25,892	28,390	29,685	29,221	27,856
Average	23,170	20,316	21,096	20,958	21,955	19,694	21,507	20,275	21,087	27,241	28,306	27,817	21,825
Average, all trucks and combinations	10,889	10,656	8,785	9,650	11,365	8,084	10,478	8,212	9,391	8,895	10,025	9,431	9,478

tables and charts in this article have been calculated by means of these trends, or by combining the actual data gathered in the summer survey with data developed from trends.

The average weights of loaded and empty trucks and truck combinations according to vehicle types are given in table 7 for each of the census regions, the main geographic regions, and for the United States as a whole in the summer of 1954. From this table it will be seen that the heaviest average weight of loaded single-unit trucks was found in New England with the Middle Atlantic region being slightly less. The heaviest average weight of truck combinations was found in the Pacific region with that in the Mountain region being slightly smaller. The highest average weight for loaded trucks and combinations of all types was in the Pacific region and the lowest in the East South Central region. The average empty weights followed a regional pattern similar to that for the average loaded weights.

Pay Load Determined

If it is assumed that the average empty weight of loaded vehicles of a given type is the same as the average weight of empty vehicles of that same type, then subtracting average empty weight from average loaded weight gives average carried load for the vehicle type. On this basis, the average loads carried in vehicles of different types and the relation of these loads to the average loaded weights of the vehicles are as shown in table 8. Thus, in general, the heavier the vehicle type the larger the proportion of the gross weight on the average that consists of carried load or pay load. The pay load for three or more axle trucks and combinations averages about one-half of the total weight of the loaded vehicle, whereas for two-axle vehicles it averages much less.

The vehicle weights of loaded and empty single-unit trucks and truck combinations on main rural roads from 1936 to 1954 are given in figure 8 as a percentage of such weights in 1950. The weights of single-unit trucks, loaded or empty, have changed little in the period of years included, being only about 10 percent greater in 1954 than in 1936. Truck-combination weights, on the other hand, have increased rather steadily throughout the period—the loaded weights being about 61 percent greater and the empty weights about 73 percent greater in 1954 than in 1936.

Travel by Loaded and Empty Trucks

The volume of travel on main rural roads by loaded and empty trucks and truck combinations is shown in figure 9 for each year from 1936 to 1954, inclusive. Probably the most interesting feature of the graph is the change in the relation between loaded and empty single-unit trucks which took place during the war years and has continued since. In 1936 almost 61 percent of the single-unit trucks were loaded. With the large expansion of hauling that occurred in 1941 as the Nation prepared for war, that figure increased to 65 percent loaded. When war developed and driving restrictions were invoked, many small truck owners found it advantageous to drive these vehicles instead of their automobiles for general transportation purposes. Thus it soon developed that the lighter-weight trucks were being used more frequently for personal transportation than had previously been the case.

This revolutionary change in the use of the smaller vehicles was carried to such an extent that in 1945 it was found that even including the heavier trucks in the general single-unit classification, these vehicles were being used for the transportation of goods on less than one-half of their travel. After the war was over, the popularity of the light-weight trucks as a means of personal transportation appears to have continued, and in 1949 only about 46 percent of their travel involved carrying a load. The corresponding figure for 1954 is 49 percent. At no time since the war has travel by loaded

Table 8.—Average carried loads by trucks and truck combinations in the summer of 1954, in relation to the average loaded weights of such vehicles

Vehicle type	Average loaded weight	Average empty weight	Average carried load	Relation of carried load to loaded weight
	Lbs.	*Lbs.*	*Lbs.*	*Pct.*
Single-unit trucks:				
Panel and pickup	5,307	4,091	1,216	22.9
Other 2-axle, 4-tire	6,582	4,976	1,606	24.4
Other 2-axle, 6-tire	14,337	8,340	5,997	41.8
3-axle	30,702	14,912	15,790	51.4
Truck combinations:				
Truck-tractor and semitrailer	42,327	21,325	21,002	49.6
Truck and trailer	59,422	27,856	31,566	53.1

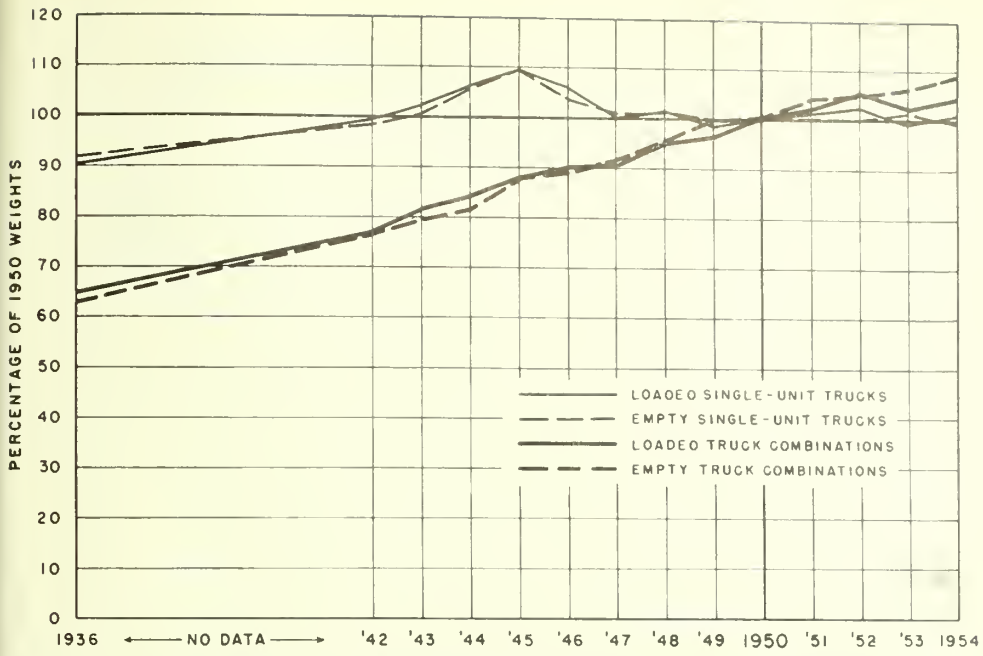

Figure 8.—Average weights of loaded and empty trucks and truck combinations on main rural roads, 1942–54 and a prewar year, as a percentage of such weights in 1950.

single-unit trucks equalled that by empty vehicles of this type.

For truck combinations, the relation between the loaded and empty vehicle-mileage of travel has been fairly uniform throughout the entire period. In 1936, 72 percent of the travel was made by loaded vehicles while in 1954 the figure was 68 percent. The vehicle-mileage of loaded single-unit trucks in 1954 was about double that in 1936, while the vehicle-mileage of loaded combinations was nearly five times that in the earlier years.

Volume of Freight Hauled

The average load carried by trucks and truck combinations on main rural roads from 1936 to 1954 is shown in figure 10 as a percentage of the amounts carried in 1950. For single-unit trucks, the average load increased about 23 percent from 1936 to 1941. Since that time loads have remained relatively constant, the curve fluctuating slightly above and below the 100-percent line on the chart. On the other hand, the average load for truck combinations increased rather steadily from 1936 to 1950 and has changed but little since then—the 1954 figure being but 2.7 percent above that for 1950. For the period 1936–54, the increase in average carried load was 29 percent for single-unit trucks and 58 percent for truck combinations. Due to the increasing use of truck combinations, the average weight of loads carried by all trucks and truck combinations had a still greater increase, amounting to 98 percent.

The ton-mileage of loads carried by trucks and truck combinations on main rural roads from 1936 to 1954 is shown in figure 11 as a percentage of amounts in 1950. From 1936 to 1950, the ton-mileage of loads carried increased 108 percent for single-unit trucks and 565 percent for truck combinations. For all trucks and combinations, the increase was 332 percent. From 1950 to 1954, the increase in ton-mileage was more gradual, and was slightly less for truck combinations than for single-unit trucks. The increases over this 4-year period were 23 percent for single-unit trucks, 17 percent for truck combinations, and 19 percent for all vehicles. For the period 1936–54, the increase was 156 percent for single-unit trucks, 681 percent for truck combinations, and 414 percent for all vehicles.

The actual ton-mileage of freight carried annually from 1936 to 1954, inclusive, by trucks and truck combinations on main rural roads is shown in figure 12. The chart shows the tremendous growth in ton-miles of freight transported by truck combinations since the beginning of the planning surveys. In 1936 an estimated 13.7 billion ton-miles were transported by these combination-type vehicles, and slightly less than 14.3 billion ton-miles were transported by single-unit trucks. By 1940, the combination vehicles were hauling more than the single-unit trucks and by 1953, the ton-mileage hauled by combinations was more than three times that by single-unit trucks. With the general slackening in business in 1954, hauling by combinations dropped slightly while that by single-unit trucks continued to rise, with the result that the ton-mileage hauled by combinations was somewhat less than three times the ton-mileage by single-unit trucks.

The growth in ton-mileage by single-unit trucks and truck combinations from 1936 to 1954 is illustrated in another manner in figure 13 (p. 242). Ton-mileage is the product of vehicle-mileage traveled by loaded vehicles and the average tonnage carried by each vehicle; this chart shows the changes that have taken place in each of these factors. The horizontal scale measures the vehicle-mileage for loaded

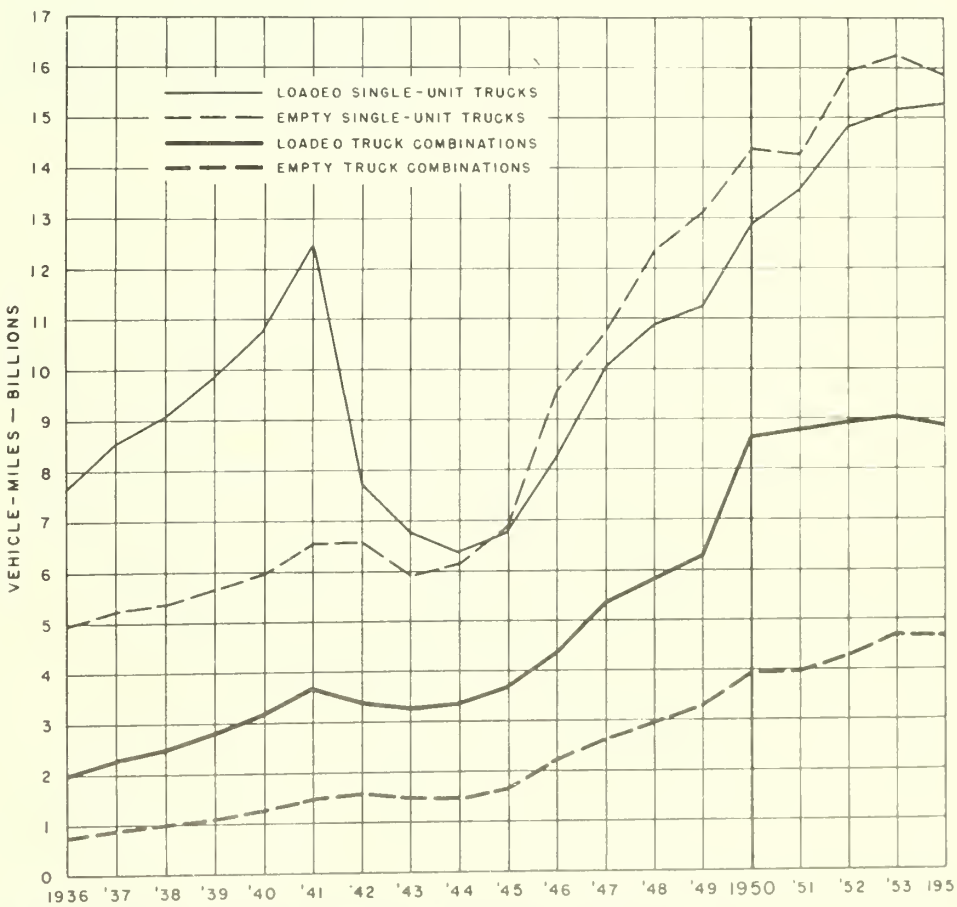

Figure 9.—Vehicle-miles of travel on main rural roads by loaded and empty trucks and truck combinations, 1936–54.

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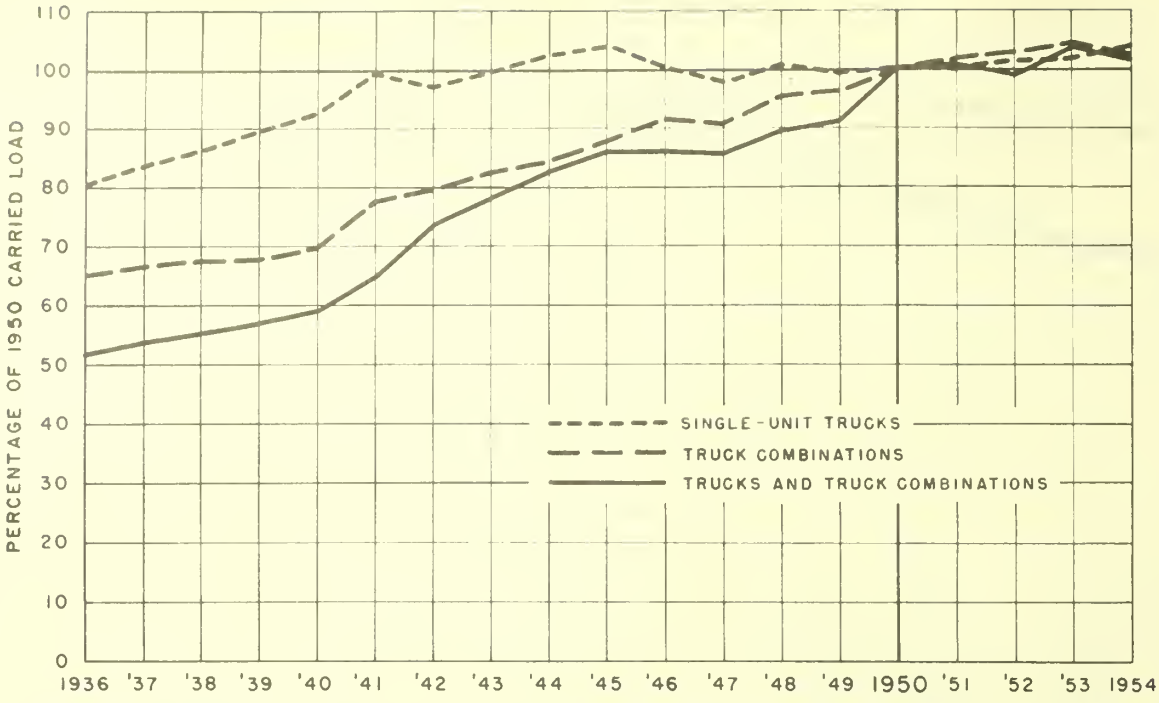

Figure 10.—Average loads carried by trucks and truck combinations on main rural roads, 1936–54, as a percentage of the amounts carried in 1950.

vehicles of each type, and the vertical scale measures the average carried load. Ton-mileage, the product of these two factors, is represented by the areas of the rectangles.

For single-unit trucks, the increase in ton-mileage from 14.3 billion in 1936 to 36.6 billion in 1954 came about mainly through an increase in the vehicle-mileage of loaded vehicles, since there was very little increase in the average carried load for this class of vehicles, especially from 1950 to 1954. For truck combinations, the increase in ton-mileage from 13.7 billion in 1936 to 107.3 billion in 1954 came about through a substantial increase in average carried load, and a much greater proportional increase in the vehicle-mileage of loaded vehicles.

It can be seen from figure 13 that almost the entire development of the movement of freight over the highways by the heavy combination-type vehicle has taken place

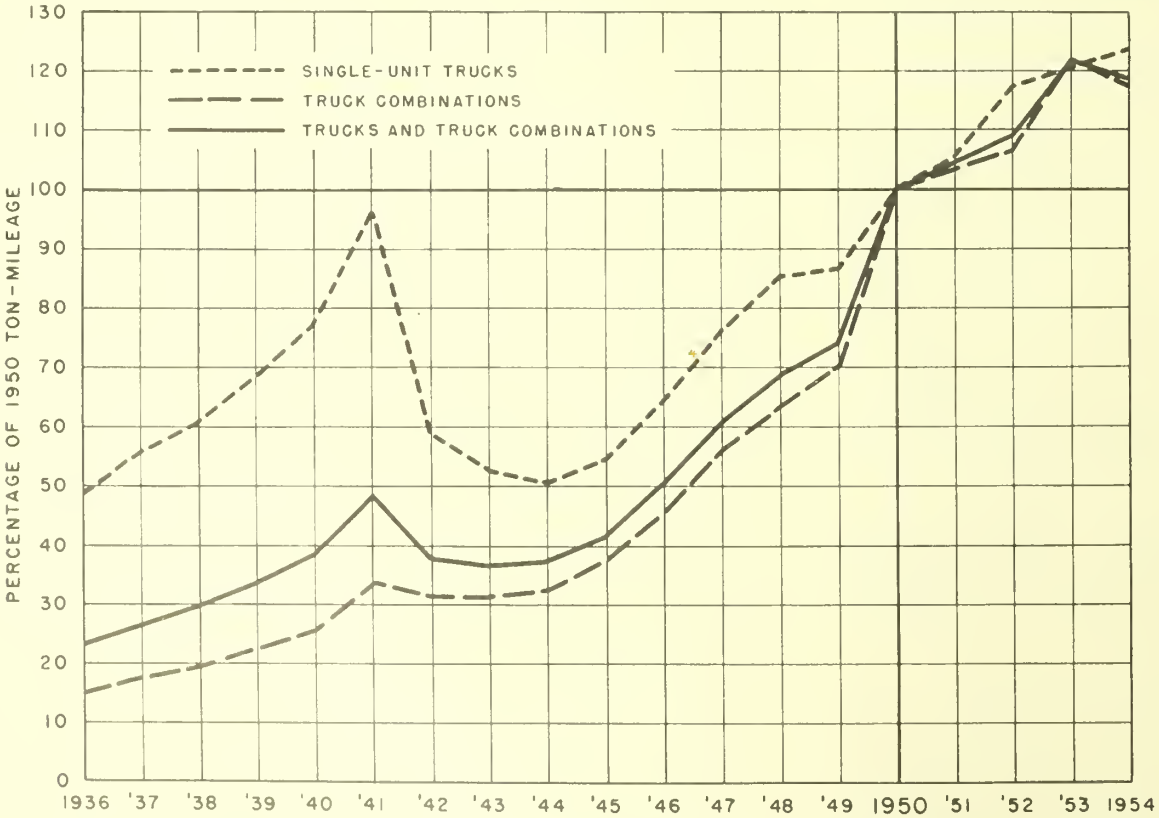

Figure 11.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–54, as a percentage of the amounts carried in 1950.

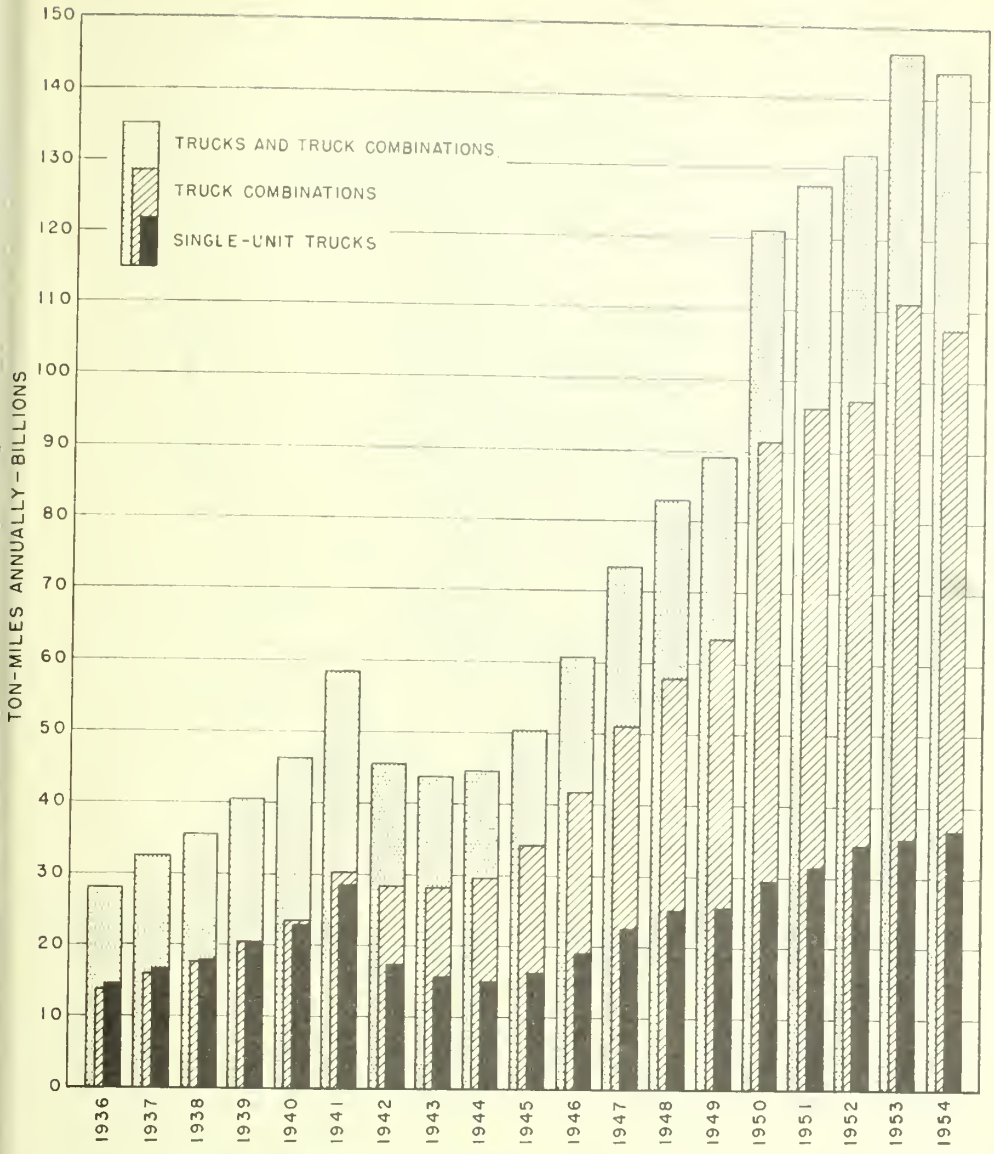

Figure 12.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–54.

since 1936. In that year, over one-half of the hauling, measured in ton-mileage, was performed by single-unit trueks; in 1954 about three-fourths of it was done by the heavy combination vehieles.

A eomparison of the estimated percentage of trueks loaded, average carried load, and ton-miles of freight carried on main rural roads in 1940, 1945, 1950, 1953, and 1954 is given separately for single-unit trueks and truek combinations in table 9. The table shows the extent to whieh the ton-mileage gains were due to increased loading per vehicle. The inereases beyond this point resulted, of course, from inereased mileage by loaded vehicles.

In eonsidering the 1954:1940 ratios on the bottom line of the table for example, it will be noted that the ton-mileage hauled by combinations was 4.6 times as great at the end of this 14-year period as it was at the beginning, while the ratio for the average weight of earried load was 1.47 and the ratio for the pereentage of vehicles loaded was 0.95. From table 4, the eorresponding ratio for the vehicle-mileage traveled by all vehicles of this type both loaded and empty was 3.29, whieh when multiplied by 0.95 gives a ratio of 3.13 for loaded vehicles. Obviously, most of the enormous increase in ton-mileage was due to increased vehicle-mileage rather than to heavier loading, though the latter factor was of considerable importance in the case of the combination-type vehiele.

For single-unit trucks, the percentage of vehicles loaded decreased more than the average load increased during the period 1940–54. In other words, there was an actual decrease in the average load for vehicles of this type when both loaded and empty vehicles were included in computing the average. The 60-percent increase in ton-mileage carried by single-unit trucks was therefore smaller than the increase in vehicle-mileage, which was 86 percent as shown by the ratio in table 4.

Comparative information in greater detail than table 9 concerning the percentage of vehicle-miles of travel, percentage loaded, the average earried load, and percentage of total ton-miles hauled on main rural roads in 1954 and 1953 is given in table 10. Many interesting comparisons may be made from these data. For instance, two-axle, six-tire, single-unit trucks, which are the principal load-carrying single-unit vehicles and account for about 28 percent of the total truck travel, carry only about 18 percent of the ton-mileage. On the other hand, truck-tractor and semi-trailer combinations account for slightly less than 30 percent of the total vehicle-mileage (about the same as the two-axle, six-tire trucks), but earry about 68 percent of the ton-mileage or almost four times the amount carried by the smaller vehicles.

The data concerning percentage loaded and average carried load are also interesting. Panels and pickups, which carry loads of three-fourths of a ton or less, travel loaded less than 40 percent of the time. Assuming that there is a return trip with no load for each trip with a load, only 80 percent of the travel can be accounted for on the basis of the hauling of goods. Since vehicles are often loaded in both directions, the use of these vehicles as a substitute for passenger cars must be well in excess of 20 percent of their total travel.

The slightly larger two-axle trucks, with single tires and carrying loads averaging

Table 9.—Comparison of estimated percentage of trucks loaded, average carried load, and ton-miles carried on main rural roads in 1940, 1945, 1950, 1953, and 1954

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1940	65.9	3.32	46,247	64.4	2.13	22,899	71.6	7.41	23,348
1945	55.1	4.84	50,365	49.6	2.40	16,187	69.2	9.31	34,178
1945: 1940 ratio	*.83*	*1.46*	*1.09*	*.77*	*1.13*	*.71*	*.97*	*1.26*	*1.46*
1950	53.9	5.64	121,091	47.2	2.31	29,645	68.5	10.62	91,446
1950: 1945 ratio	*.98*	*1.17*	*2.40*	*.95*	*.96*	*1.83*	*.99*	*1.14*	*2.68*
1950: 1940 ratio	*.82*	*1.70*	*2.62*	*.73*	*1.08*	*1.29*	*.96*	*1.43*	*3.92*
1953	54.6	5.82	146,810	48.2	2.35	35,602	68.2	11.07	111,208
1953: 1950 ratio	*1.01*	*1.03*	*1.21*	*1.02*	*1.02*	*1.20*	*.99*	*1.04*	*1.22*
1953: 1945 ratio	*.99*	*1.20*	*2.91*	*.97*	*.98*	*2.20*	*.98*	*1.19*	*3.25*
1953: 1940 ratio	*.84*	*1.75*	*3.17*	*.75*	*1.10*	*1.55*	*.95*	*1.49*	*4.76*
1954	55.1	5.74	143,901	49.1	2.40	36,559	67.9	10.91	107,342
1954: 1953 ratio	*1.01*	*.99*	*.98*	*1.02*	*1.03*	*1.03*	*1.00*	*.99*	*.97*
1954: 1950 ratio	*1.02*	*1.02*	*1.19*	*1.04*	*1.04*	*1.23*	*.99*	*1.03*	*1.17*
1954: 1945 ratio	*1.00*	*1.19*	*2.86*	*.99*	*1.00*	*2.26*	*.98*	*1.17*	*3.14*
1954: 1940 ratio	*.84*	*1.73*	*3.11*	*.76*	*1.13*	*1.60*	*.95*	*1.47*	*4.60*

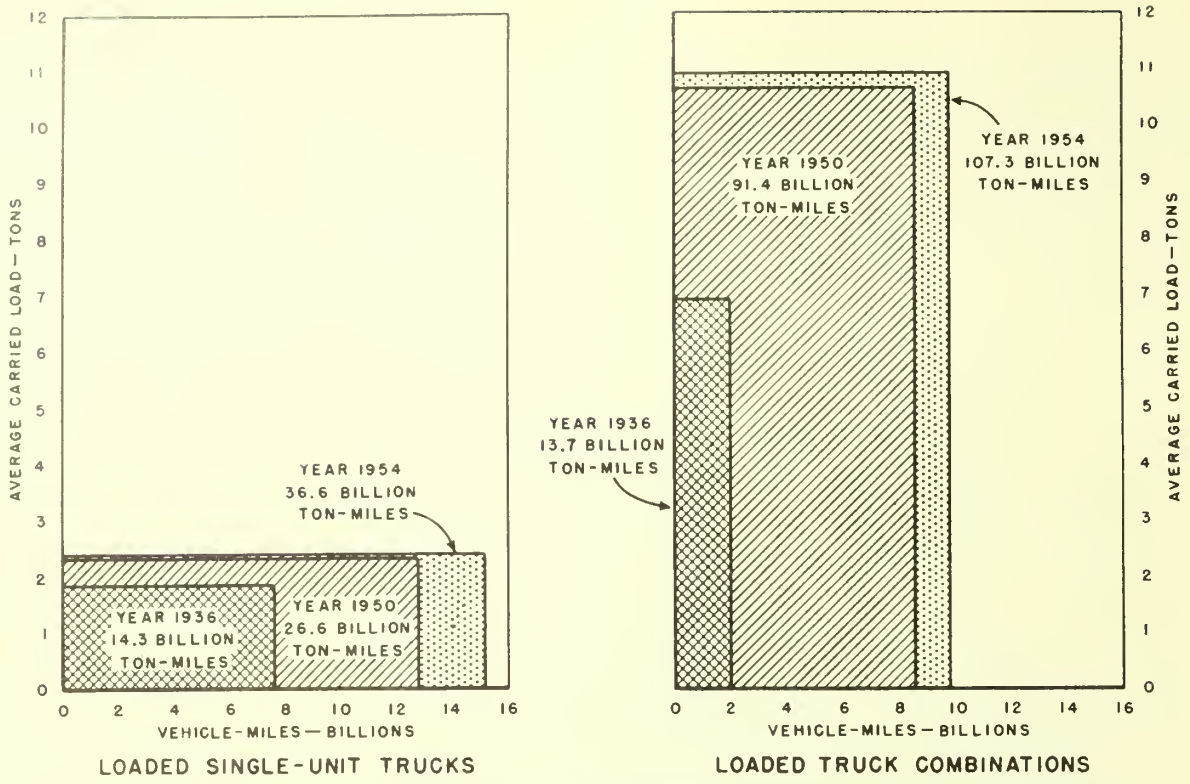

Figure 13.—Vehicle-miles of travel, average carried load, and ton-miles carried by trucks and truck combinations on main rural roads in 1954 compared with 1936 and 1950.

around one ton, are loaded about one-half of the time indicating still some use for personal transportation, while the larger single-unit vehicles operate with a load almost 60 percent of the time. Many of the truck combinations are common carriers which seldom are operated empty, inasmuch as they continually are picking up and discharging freight. They are found loaded, therefore, in larger proportion than the large single-unit trucks. Many of the latter type carry loads of a one-way variety, such as hauling building materials to a construction project.

Shift to Two-Axle Semitrailers

The percentage of vehicle-mileage traveled by trucks of various types in 1954 compared with 1950 is shown in figure 14. An interesting fact shown by this chart is the great shift that took place in this short period of time from single-axle semitrailers to those equipped with dual axles. In the earlier period, 18.5 percent of the truck and truck combination travel was made by two-axle tractors pulling single-axle semitrailers, while travel by the same type of tractor with dual-axle semitrailers amounted to only 9.0 percent.

The 1954 survey figures indicate that this relation has changed materially, and the percentage for the dual-axle semitrailers now exceeds the percentage for the single-axle semitrailers. Considerable expansion in the use of three-axle tractors pulling semitrailers equipped with dual axles likewise may be noted, though the percentage for this vehicle type is still relatively low for the country as a whole. These vehicles still are found principally in the western regions, although more are being used elsewhere especially in the north central regions. The shift from one-axle to two-axle semitrailers has taken place in order that the maximum possible pay load can be carried under the weight restrictions in effect in some States.

The percentage of ton-miles hauled by various truck types in 1954 compared with 1950 is shown in figure 15. This chart emphasizes, even more than figure 14, the shift from three-axle tractor-semitrailer combinations to four- and five-axle combinations between 1950 and 1954. In 1954, the four-axle combinations accounted for a larger portion of the ton-mileage than any other single vehicle type, whereas in 1950 the three-axle combinations predominated.

Heavy Gross-Load Frequencies

The frequency of gross loads of 30,000 40,000, and 50,000 pounds or more per 1,000 trucks and truck combinations on main rural roads in the summer of 1954 is shown for the different census regions in figure 16. This chart shows that the greatest frequency of such heavy vehicles is found in the Pacific region. In that area in 1954, out of each 1,000 trucks and combinations including empties, 238 weighed 30,000 pounds or more, 177 weighed 40,000 pounds or more, while 149 weighed 50,000 pounds or more. The East North Central region ranked second for vehicles weighing over 30,000 pounds. Here it was found that for each 1,000 trucks and combinations, 230 weighed that amount or more, while 145 weighed 40,000 pounds or more, and 75 weighed 50,000 pounds or more. The West North Central region ranked third for vehicles weighing 30,000 pounds or more but ranked second for vehicles weighing 40,000 pounds and 50,000 pounds or more.

Table 10.—Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried on main rural roads in 1954 compared with 1953

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1954	1953	1954	1953	1954	1953	1954	1953
					Tons	*Tons*		
Single-unit trucks:								
Panel and pickup	34.41	33.49	39.8	39.7	0.71	0.70	3.08	2.89
Other 2-axle, 4-tire	3.00	2.96	48.7	52.3	.89	1.03	.41	.50
Other 2-axle, 6-tire	28.28	29.02	59.7	58.3	3.36	3.25	17.93	17.18
3-axle	2.50	2.37	58.2	58.0	8.63	8.54	3.99	3.66
All single-unit trucks	68.19	67.84	49.1	49.0	2.40	2.34	25.41	24.23
Truck combinations:								
Truck-tractor and semitrailer	29.85	29.85	68.4	68.6	10.63	10.67	68.67	68.00
Truck and trailer	1.96	2.31	60.1	65.7	15.90	16.43	5.92	7.77
All truck combinations	31.81	32.16	67.9	68.4	10.91	11.07	74.59	75.77
All trucks and combinations	100.00	100.00	55.1	55.1	5.74	5.83	100.00	100.00

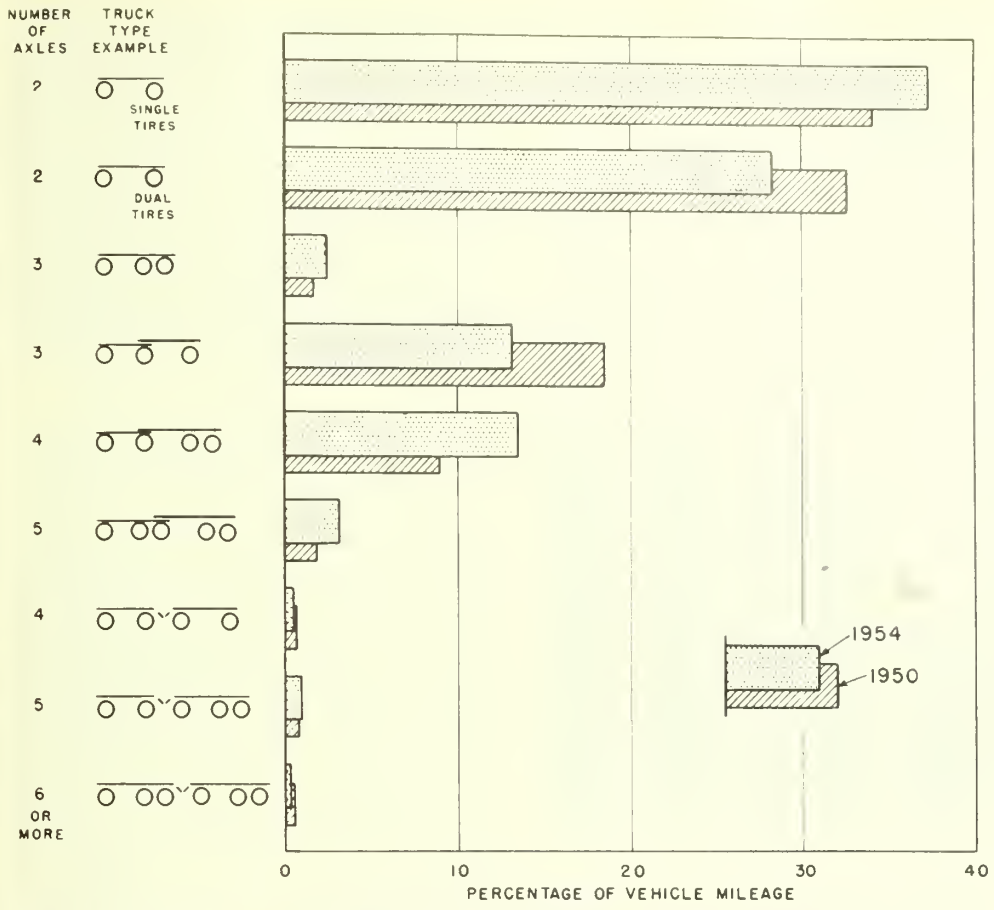

Figure 14.—Percentage of vehicle-mileage traveled by various types of trucks and truck combinations on main rural roads in 1954 compared with 1950.

weighing 30,000 pounds or more and of those weighing 40,000 pounds or more were slightly lower in 1954 than in 1953, the number of vehicles weighing 50,000 pounds or more reached an alltime high in 1954. Since the beginning of the planning surveys in the 1936–37 period, the frequency of loads of 30,000 pounds or more increased almost 350 percent, those of 40,000 pounds or more increased over 1,000 percent, and those of 50,000 pounds or more increased more than 2,000 percent.

The 1954 gross-weight frequency data by vehicle type and region are presented in table 11. Since no panels, pickups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, there is no entry for these vehicles in the table. They are included, however, in the total number of vehicles weighed in computing the frequencies for all single-unit trucks and for all trucks and combinations.

As was noted in the discussion concerning figure 16, heavy gross weights are more frequent in the Pacific region than in other parts of the country. In this region, 238 of each 1,000 trucks and truck combinations weighed 30,000 pounds or more, while 731 of each 1,000 truck combinations weighed that much. Since about 21 percent of these vehicles in the Pacific region were empty, the number loaded per thousand was 790 and it is obvious that most of these weighed more than 30,000 pounds. In fact, a number of empty vehicles of this type weighed over 30,000 pounds and a few over 40,000 pounds.

Frequencies of 221, 148, and 93 were found in that area for the 30,000-, 40,000-, and 50,000-pound weight classes, respectively. The lowest frequencies of these heavy loads were found in the East South Central region where 145, 61, and 18, respectively, were found for the three groups.

Figure 17 shows the number of heavy gross weights per 1,000 loaded and empty trucks and truck combinations on main rural roads in the summers of 1942–54 and in a prewar year (1936–37 period). This chart shows the yearly variations in the frequencies of vehicles weighing 30,000, 40,000, or 50,000 pounds or more. During this period of about 18 years the trend of the frequency of vehicles weighing 50,000 pounds or more was rather consistently upward, and the frequency of those weighing 40,000 pounds or more was generally upward with temporary drops in 1947, 1952, and 1954. The long-range trend in frequency of weights of 30,000 pounds or more was also upward, but there was an important decline from 1945 to 1947. This decline proved to be quite temporary, however, and was followed by a rapid rise to 1950.

Since 1950 the frequency has fluctuated slightly up and down without changing significantly. While the frequencies of vehicles

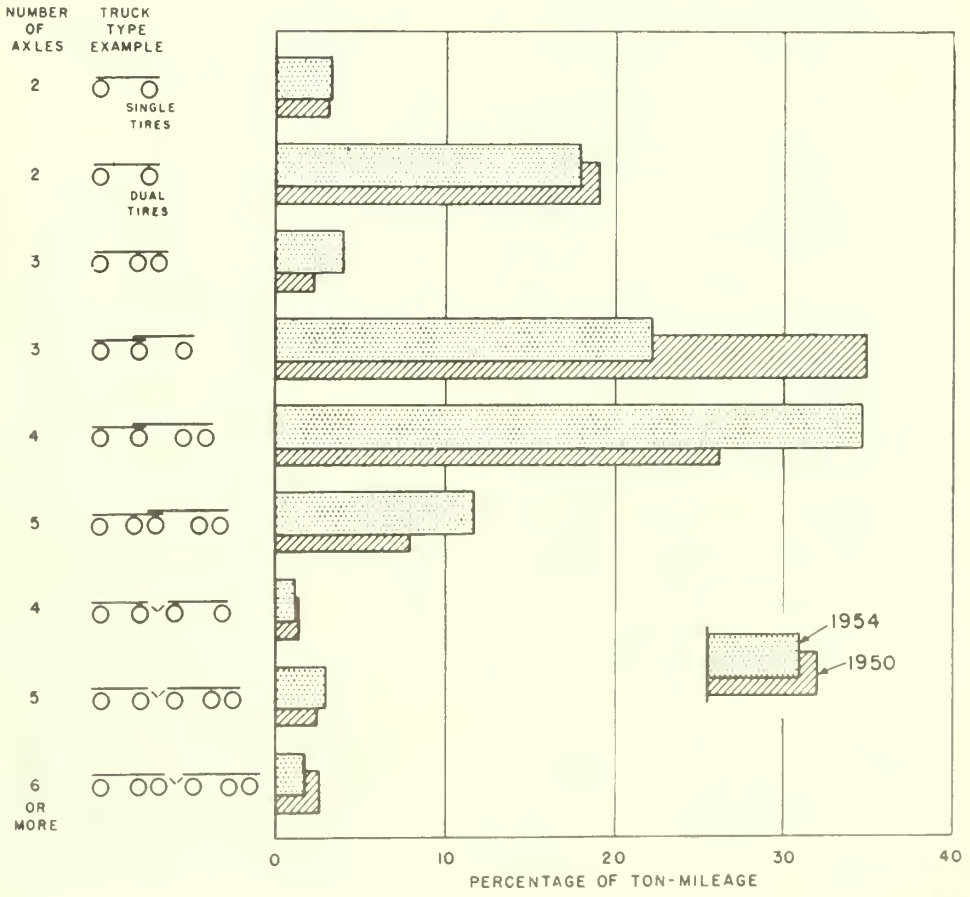

Figure 15 **(Right).**—*Percentage of ton-mileage hauled by various types of trucks and truck combinations on main rural roads in 1954 compared with 1950.*

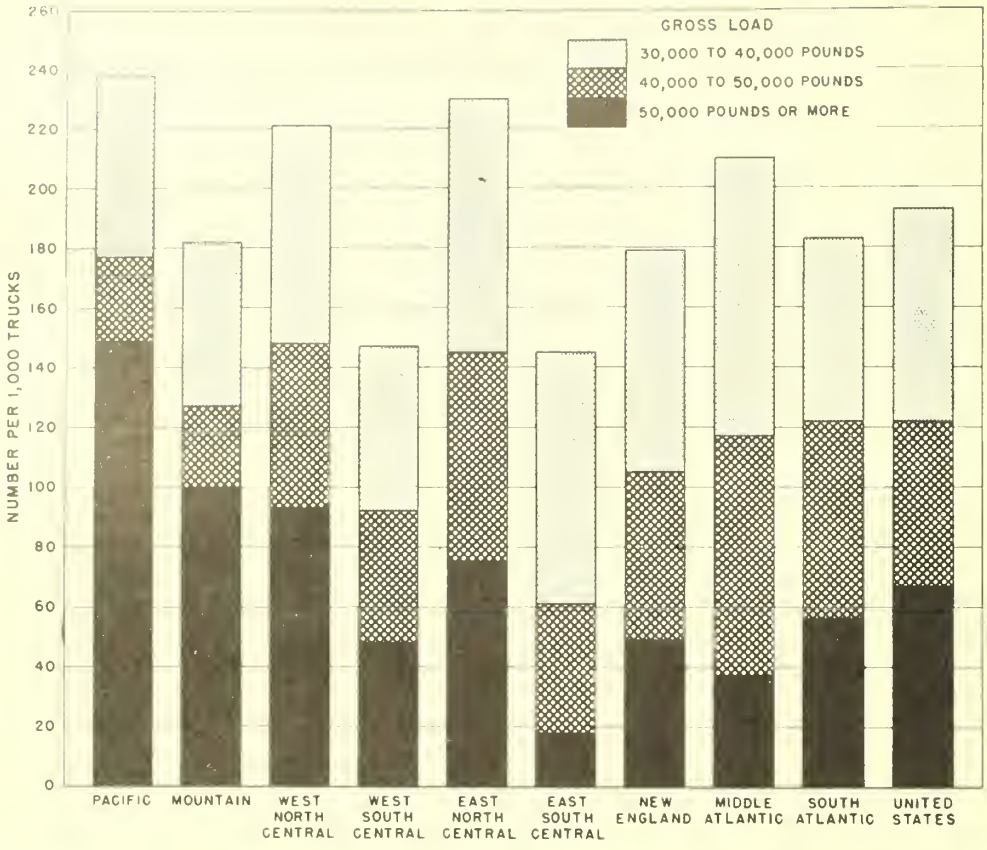

Figure 16 (Above).—***Number of gross loads of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1954, by regions.***

Other areas such as the Mountain and East South Central regions had extremely high frequencies of truck combinations weighing 30,000 pounds or more, which indicates that most of such vehicles weighed at least 30,000 pounds when loaded. The table also shows that almost one-half of the truck combinations found on the roads of the Pacific or Mountain regions weighed 50,000 pounds or more. High frequencies for heavy vehicles of the combination type are found in the western regions, while the heavier single-unit trucks are found most frequently in the eastern regions, especially New England.

Vehicle-miles of travel by trucks and truck combinations weighing 30,000, 40,000, and 50,000 pounds or more are shown in figure 18 for the summers of 1936 and 1942–54, as a percentage of such travel in 1950. This chart is based on the actual recurrence of these heavy loads in the traffic stream, and includes changes in traffic volume as well as in the frequencies of heavy loads per thousand vehicles. It shows that the vehicle-miles traveled by trucks and combinations, weighing 30,000 pounds or more, was over 1,000 percent greater in 1954 than in the prewar year, that the travel of those weighing 40,000 pounds or more was over 3,000 percent greater, and that the travel of those weighing 50,000 pounds or more was over 6,000 percent greater.

The rate of increase was especially great from 1949 to 1950, and since then has been materially less. As was seen in figure 17, the frequency of heavy gross loads per thousand vehicles has been fairly stable since 1950, and the upward slope of the curves in figure 18 from 1950 to 1953 is due principally to increased vehicle-mileage.

Heavy Axle-Load Frequencies

The number of axle loads of 18,000, 20,000, and 22,000 pounds or more per 1,000 trucks and truck-combinations in the summer of 1954 is given in figure 19 for the main rural roads of each census region. The most noticeable feature of the chart is the very high frequency of heavy axle loads in all three weight categories in the New England and Middle Atlantic regions and, to a lesser extent, in the South Atlantic region. The axle-load limits in most States in these areas are higher than those generally in effect elsewhere. In the New England region, for instance, Connecticut, Massachusetts, and Rhode Island permit axle loads up to 22,400 pounds; Maine and

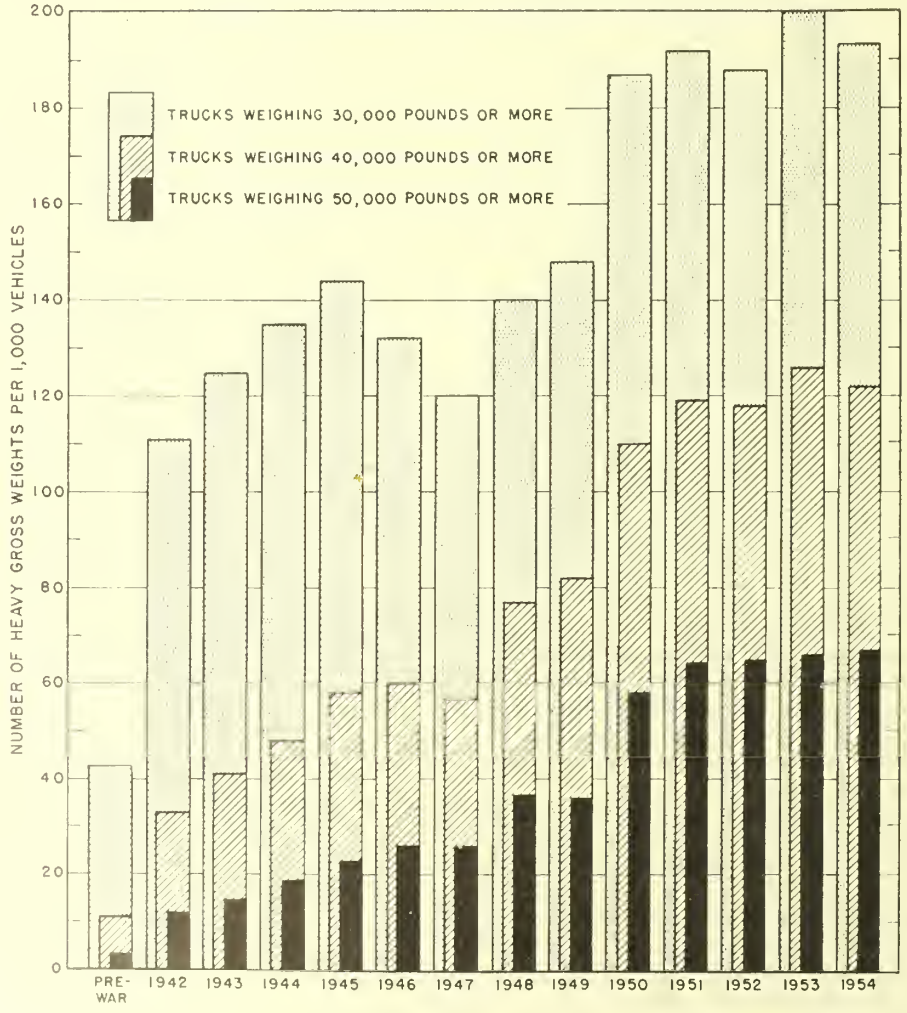

Figure 17 (Right).—***Number of gross loads of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summers of 1942–54 and a prewar year.***

Table 11.—Frequency of heavy vehicles of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1954 by main vehicle types and by regions

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Number per 1,000 Weighing 30,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	27	14	2	10	(1)	1	-----	-----	(1)	1	(1)	1	3
3-axle	507	365	251	311	313	252	324	405	309	340	200	251	296
Average	36	21	14	20	12	8	11	4	8	13	12	12	12
Truck combinations:													
Truck-tractor and semitrailer	578	516	583	545	510	671	611	544	561	694	729	715	577
Truck and trailer	-----	(2)	-----	(2)	522	-----	409	209	455	734	735	735	623
Average	577	515	579	553	511	671	602	537	557	702	731	720	580
Average, all trucks and combinations	179	210	183	192	230	145	221	147	187	182	238	214	193
Comparative average, 1953	165	199	200	196	259	129	197	135	188	183	230	211	200
Comparative average, 1950	137	221	177	189	251	102	142	146	170	160	289	233	187
Number per 1,000 Weighing 40,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	6	(1)	(1)	1	-----	1	-----	-----	(1)	(1)	-----	(1)	(1)
3-axle	238	60	59	81	48	20	31	72	39	134	11	55	59
Average	14	3	3	4	2	1	1	1	1	5	1	3	2
Truck combinations:													
Truck-tractor and semitrailer	358	303	401	356	330	291	427	346	349	494	575	542	375
Truck and trailer	-----	(2)	(2)	(2)	377	-----	172	88	293	537	524	528	436
Average	358	303	399	355	333	291	416	340	348	502	561	539	378
Average, all trucks and combinations	105	117	122	118	145	61	148	92	114	127	177	155	122
Comparative average, 1953	98	112	125	117	179	56	131	84	120	128	182	159	126
Comparative average, 1950	78	135	95	109	140	45	82	79	95	106	214	167	110
Number per 1,000 Weighing 50,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	-----	-----	(1)	(1)	-----	-----	-----	-----	-----	-----	-----	-----	(1)
3-axle	92	29	1	19	3	-----	4	-----	2	8	6	7	10
Average	4	1	(1)	1	(1)	-----	(1)	-----	(1)	(1)	(1)	(1)	(1)
Truck combinations:													
Truck-tractor and semitrailer	176	95	187	148	160	87	267	183	178	384	472	436	199
Truck and trailer	-----	(2)	-----	(2)	354	-----	129	70	266	494	469	476	393
Average	176	96	186	147	171	87	261	180	181	406	471	446	211
Average, all trucks and combinations	49	37	56	48	75	18	93	48	59	100	149	127	67
Comparative average, 1953	33	34	57	46	95	11	76	40	61	100	130	118	66
Comparative average, 1950	24	63	28	41	78	7	44	34	47	76	176	133	58

[1] Less than 5 per 10,000.
[2] Data omitted because of insufficient sample.

New Hampshire limit them to 22,000 pounds; while Vermont has no axle-load limit. A low gross-weight limitation in Vermont, however, restricts axle loading. In the Middle Atlantic region, New Jersey and New York permit axle weights up to 22,400 pounds, while Pennsylvania limits them to 20,000 pounds. Axle loads heavier than 18,000 pounds are also permitted in most States of the South Atlantic region, although Virginia, Georgia, and West Virginia have the 18,000-pound limit. In Ohio, axle loads up to 19,000 pounds are legal; all other States limit them to 18,000 pounds.

Since higher limits generally are allowed in the three regions mentioned, only a small portion of the heavy axle loads shown in these three areas are in excess of State laws, whereas in the other regions practically all axle weights over 18,000 pounds are in violation of the laws. When a comparison is made on the basis of conformity to State law, as will be discussed later, it will be seen that the rate of violation in all three of the eastern regions actually is very low. The low frequency of heavy axle loads in the Pacific region, which was shown in figure 16 to have the highest frequency of heavy gross loads, is noteworthy. Here again, the answer can be found in the State laws which encourage the distribution of the load to more axles.

The frequency of axle loads weighing 18,000, 20,000, and 22,000 pounds or more per 1,000 trucks and truck combinations on main rural roads in the summers of 1942–54 and in a prewar year is shown in figure 20. A most important fact indicated by the chart is the reversal in trend that began in 1951 and has continued each year. The frequency of axles weighing 18,000 pounds or more in 1950 was over seven times that in the prewar year, but from the 1950 high there was a consistent drop each year which amounted to 35 percent by 1954. Likewise, the frequency of 20,000-pound axles in 1950 was almost eight times that in the prewar year, but by 1954 the frequency of loads in this category was less than one-half of that in 1950. The number of axle loads of 22,000 pounds or more increased ninefold from the prewar year to 1950, but dropped more than 60 percent below that peak by 1954.

Load Distribution Improves

The rapid decrease in the frequency of heavy axle loads in the face of a stationary or slightly increasing number of heavy gross weights, as shown in figure 17, may at first appear illogical. However, the reason for the differences in the two charts was clearly brought out in the preceding discussion of figures 14–15. Many truckers are replacing their single-axle semitrailers with tandem-axle types, and the loads are therefore distributed over a larger number of axles. The net result of this equipment change is that many heavy loads, which formerly caused one or more axles of a vehicle to be overweight, are now transported in vehicles that are so designed that the loads are better distributed and no weight restriction is exceeded. Such changes, notwithstanding liberalization of the legal weight restrictions in some States, appear to be the main reason for the gratifying improvement in weight distribution that has occurred since 1950.

The number of axles weighing 18,000, 20,000, and 22,000 pounds or more for each 1,000 loaded and empty trucks and truck combinations on main rural roads in the summer of 1954 are given in table 12 (p. 248) for the main vehicle types, by census regions, and by the main geographic regions. Since no two-axle, single-unit trucks with single tires were found with axles weighing as much as 18,000 pounds, that type is not mentioned

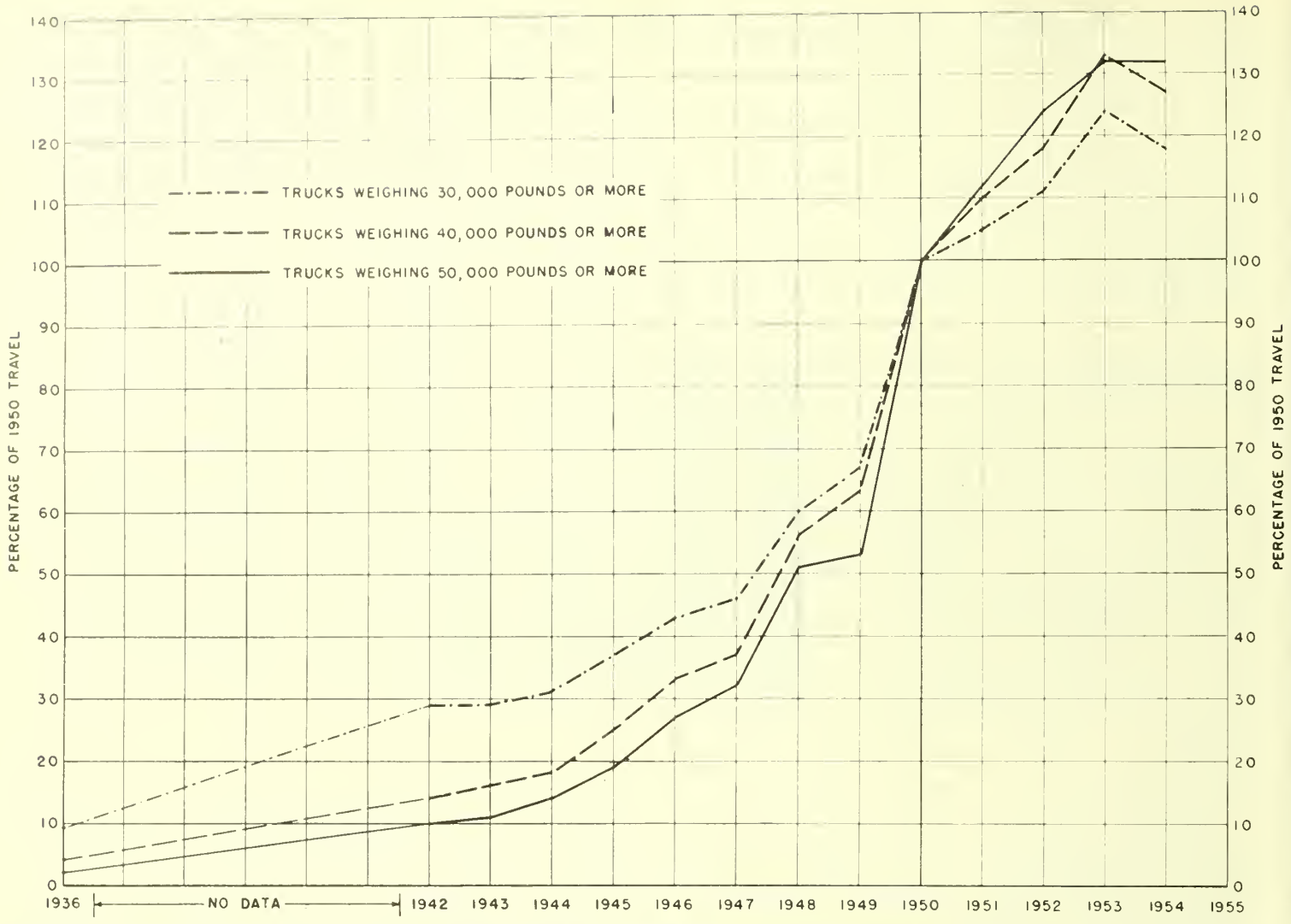

Figure 18.—Travel of trucks and truck combinations weighing 30,000, 40,000, and 50,000 pounds or more on main rural roads in the summers of 1942–54 and a prewar year, as a percentage of such travel in 1950.

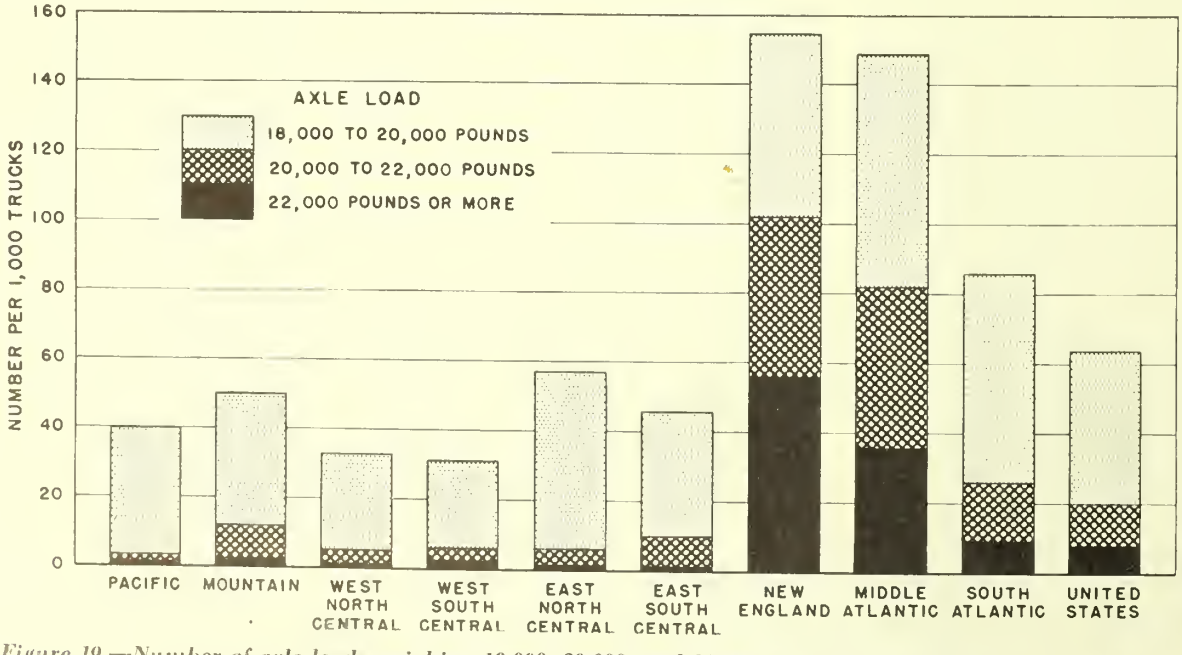

Figure 19.—Number of axle loads weighing 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads in the summer of 1954, by regions.

in the table. The number of such vehicles counted is included, however, in obtaining the total frequency of all single-unit trucks and of all trucks and combinations.

The comparisons with 1953 and 1950 at the bottom of each section of the table show the trend of the frequency for each weight category. For instance, the number of axle loads weighing 18,000 pounds or more in 1954 compared with the previous year was downward in the New England, South Atlantic, East North Central, West South Central, and Pacific regions. In the remaining regions, the 1954 frequencies are above those for 1953, but with the exception of the New England region, are well below those of 1950. A comparison of the data in the other weight categories shows similar trends. For the United States as a whole, the trend is consistently downward for all axle weight levels. It is obvious that, for all regions and the three weight categories, it is the combination-type vehicles rather than the single-unit trucks that produce the greatest frequency of heavy axle loads.

In order to give a clearer picture of what is happening on the roads, figure 21 presents travel of vehicles with axles weighing 18,000, 20,000, and 22,000 pounds or more, in the summers of 1942–54 and a prewar year, as a

Figure 20 (Right).—*Number of axles weighing 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads in the summers of 1942–54 and a prewar year.*

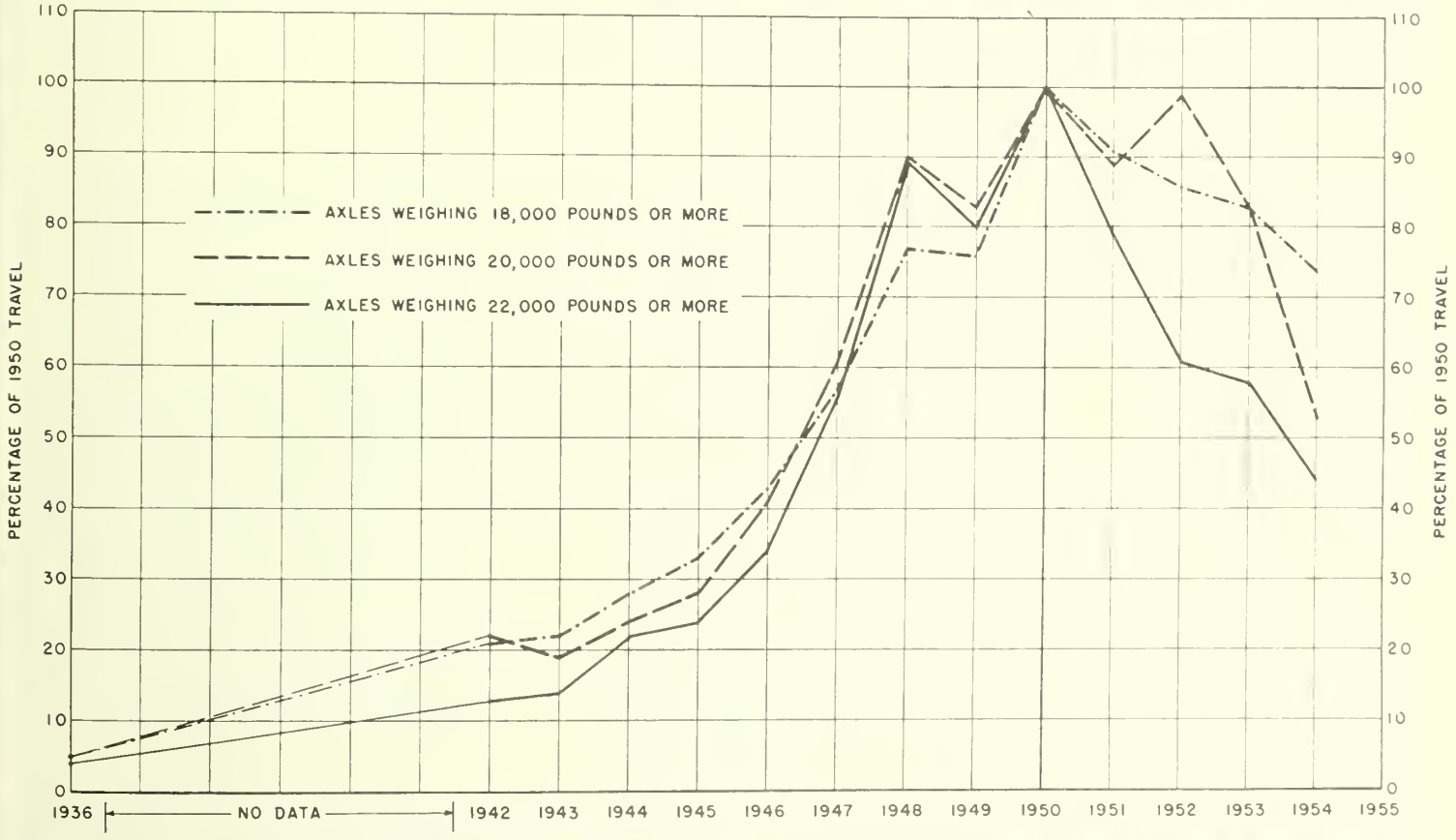

Figure 21.—Travel of vehicles with axles weighing 18,000, 20,000, and 22,000 pounds or more in the summers of 1942–54 and a prewar year, as a percentage of such travel in 1950.

Table 12.—Frequency of heavy axles of 18,000, 20,000, and 22,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1954 by main vehicle types and by regions

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Number per 1,000 Weighing 18,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	60	51	33	41	14	33	9	16	17	27	34	31	27
3-axle	311	73	77	104	9	21	8	45	16	70	33	46	57
Average	45	31	18	24	7	15	4	6	8	11	12	12	13
Truck combinations:													
Truck-tractor and semitrailer	465	341	247	307	115	164	83	98	111	154	94	119	176
Truck and trailer	------	(1)	------	(1)	229	------	111	44	178	242	118	155	160
Average	465	339	245	306	122	164	84	97	113	171	101	128	175
Average, all trucks and combinations	155	149	86	115	57	46	33	31	42	50	40	45	64
Comparative average, 1953	161	137	104	123	67	39	29	32	44	43	41	42	69
Comparative average, 1950	137	208	100	147	98	63	45	67	72	83	69	75	96
Number per 1,000 Weighing 20,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	41	29	11	22	1	7	2	4	3	7	1	4	10
3-axle	168	61	8	40	------	3	------	------	1	18	(2)	7	18
Average	29	18	5	12	(2)	3	2	1	1	3	(2)	1	5
Truck combinations:													
Truck-tractor and semitrailer	307	170	77	138	13	37	10	17	17	37	7	19	56
Truck and trailer	------	(1)	------	(1)	4	------	12	------	5	51	9	21	16
Average	306	169	76	137	12	37	10	17	16	40	8	20	54
Average, all trucks and combinations	102	82	26	53	6	10	5	6	6	12	3	7	20
Comparative average, 1953	93	68	44	58	11	6	3	7	9	9	3	4	28
Comparative average, 1950	82	131	38	80	22	19	12	23	19	35	16	24	39
Number per 1,000 Weighing 22,000 Pounds or More													
Single-unit trucks:													
2-axle, 6-tire	28	9	5	10	(2)	1	1	1	1	1	(2)	1	4
3-axle	101	36	2	23	------	3	------	------	1	2	------	1	9
Average	19	6	2	6	(2)	(2)	(2)	(2)	(2)	(2)	(2)	(2)	2
Truck combinations:													
Truck-tractor and semitrailer	158	85	26	63	2	3	1	5	2	9	3	5	23
Truck and trailer	------	------	------	------	------	------	12	------	3	------	3	2	2
Average	158	84	26	63	2	3	1	5	2	7	3	5	21
Average, all trucks and combinations	56	36	9	24	1	1	1	2	1	2	1	2	8
Comparative average, 1953	46	37	15	26	2	2	(2)	2	1	2	1	1	9
Comparative average, 1950	39	80	13	42	7	5	3	9	6	16	3	9	18

[1] Data omitted because of insufficient sample.
[2] Less than 5 per 10,000.

percentage of such travel in 1950. These curves show that when vehicle-mileage trends are considered, there is a steady upward trend in heavy axle-load frequencies in all years from 1936 to 1950 with the exception of 1949. Since 1950, however, the general trend in heavy axle-weight frequencies has been downward, notwithstanding the general upward trend in travel and in heavy gross-load frequencies that prevailed during the period. Thus by a shift to vehicles with a larger number of axles, the truckers are hauling more and bigger loads over the highways and still subjecting them to less frequent applications of heavy and destructive axle loads.

Loads Exceeding Legal Limits

Figure 22 shows the number of trucks and truck combinations for each 1,000 loaded and empty vehicles that exceeded the permissible axle, axle group, or gross-weight legal limits in effect in the State of observation during the summer of 1954. As might be expected, many vehicles exceeded more than one of the State limits and some of the larger vehicles or combinations had more than one axle loaded in excess of the limit. The data on which this chart is based were obtained by counting each vehicle only once, regardless of the number of ways in which it was overloaded. The chart gives an indication of the degree of compliance with State laws in each of the census regions.

In the New England region, for example, where there was a high frequency of heavy axle loads as shown by figure 19, vehicles were mostly within the legal limits of from 20,000 to 22,400 pounds. Only 4.1 percent of those sampled in 1954 violated the weight laws to any extent, and only 0.7 percent exceeded the limits by 20 percent or more. In the Pacific region, the legal axle-load limit conforms to the 18,000-pound recommendation of the American Association of State Highway Officials, and axle-group and gross-load limits are somewhat higher than those recommended by the Association. Here 7.8 percent of the vehicles exceeded one or more of the State weight restrictions to some extent, while only 0.2 percent exceeded those limits by 20 percent or more. In the Mountain region where maximum restrictions similar to those found in the Pacific region were in effect, 6.2 percent of the vehicles were overloaded to some extent and 0.7 percent by 20 percent or more.

Tolerances have been adopted in many States which are not taken into consideration in these figures. If a tolerance of 5 percent were universally allowed, almost one-half of the violators would be excused and the average rate of punishable violation in the United States would drop from about 5 percent to a little over 2.5 percent.

The number of trucks and truck combinations in each 1,000 loaded and empty vehicles that exceeded any of the axle, axle-group, or gross-weight legal limits in effect in the United States is shown in figure 23 for the years 1947–54. This chart is interesting in that, while it shows little if any trend in the frequency of overloads of 5 percent or less, there appears to be a real trend downward in the higher percentages of overload especially since 1950. Overloads of 50 percent or more were not found in appreciable numbers in 1952 and subsequent years, and the downward trend of overloads of 20 to 30 percent and of 30 to 50 percent after 1950 is very noticeable.

Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State

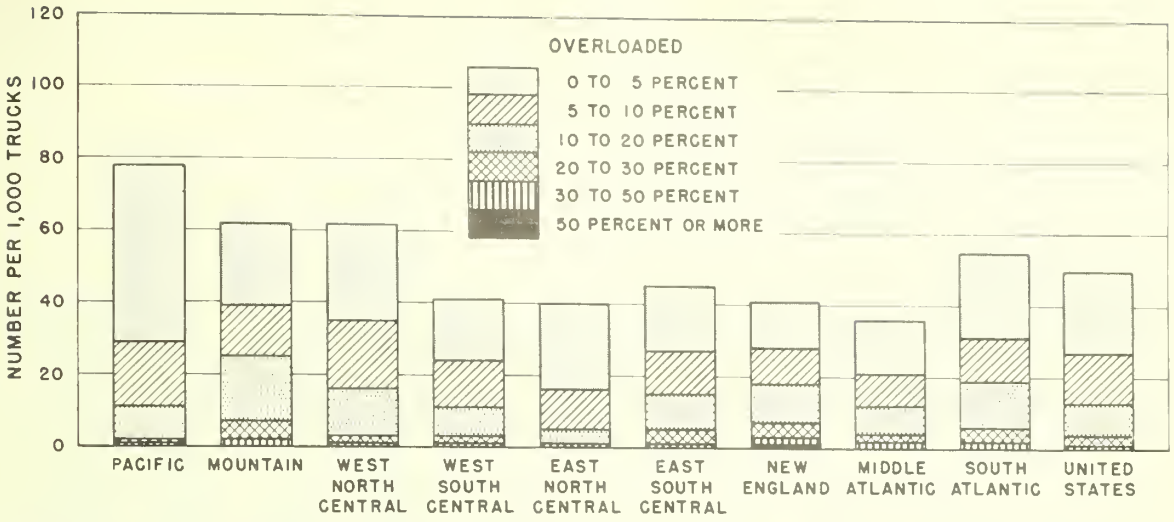

Figure 22.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in 1954, by regions.

Highway Officials and recommended to the State governments for adoption.[5] This policy recommends that no axle shall carry a load in excess of 18,000 pounds, and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles. In preparing data concerning the number of vehicles exceeding the recommendations, each vehicle was counted only once regardless of the number of ways it might exceed these standards.

The number of vehicles in each 1,000 trucks and truck combinations that, in the summer of 1954, exceeded any of the weight limits recommended by the AASHO is shown by regions in figure 24. Since the data depicted in this chart are related to a common base, meaningful comparisons concerning regional heavy loading practices are apparent. This could not be readily determined from figure 22, which was based on different laws as discussed.

In the West North Central region where the State restrictions conform closely to the AASHO recommendations, both charts show that 62 vehicles out of each 1,000 were overloaded. That number is 24 percent above the national average when compared with State legal limits (fig. 22), but 16 percent below the national average with respect to the AASHO recommendations as shown in figure 24. In the Pacific region where State axle-group and gross-weight limits are somewhat more liberal than those recommended by the AASHO, 78 vehicles from each 1,000 sampled exceeded the State limits while 142 vehicles exceeded the AASHO recommendations. On the other hand, in the East South Central region, where State laws tend to be somewhat more restrictive than the recommendations, 45 vehicles in each 1,000 exceeded the State limits while only 42 such vehicles exceeded the recommendations.

Figure 24 also shows that the Pacific region leads all other areas in the frequency of heavy loads per thousand in excess of the AASHO recommendations, and is followed in descending order by New England (108) and the East North Central region (93). However, most of the overloads in the Pacific region were only a small percentage over the AASHO recommended limits. Here there were no loads as much as 50 percent over these limits and only 8 per thousand as much as 20 percent over. In the New England region, on the other hand, 50 vehicles per thousand were 20 percent or more over the AASHO recommended limits and 6 per thousand were over 50 percent or more.

The number of vehicles in each 1,000 trucks and truck combinations that exceeded any of the weight limits recommended by the AASHO is shown in figure 25 for the years 1947–54. The number of loads in excess of the recommendations to any degree vary from 28 percent in 1947 to 48 percent in 1954 above the similar frequencies for weights in excess of State laws shown in figure 23. Such a difference is to be expected for almost every year one or more States liberalize their motor-vehicle laws, thereby allowing more heavily loaded vehicles to pass legally over the roads. In New England, for instance,

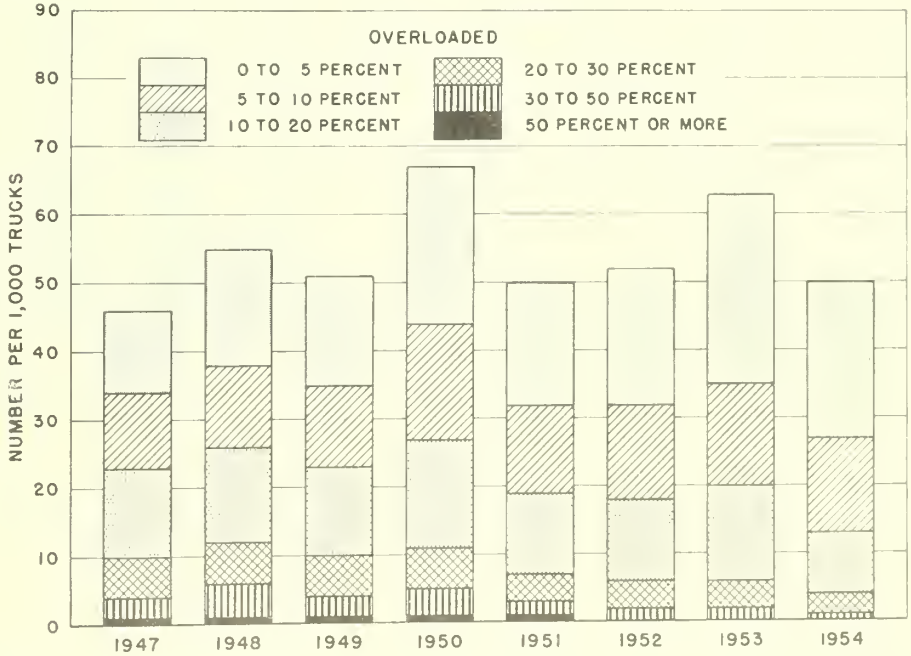

Figure 23.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the permissible axle, axle-group, or gross-weight legal limits in effect in the summers of 1947–54.

[5] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

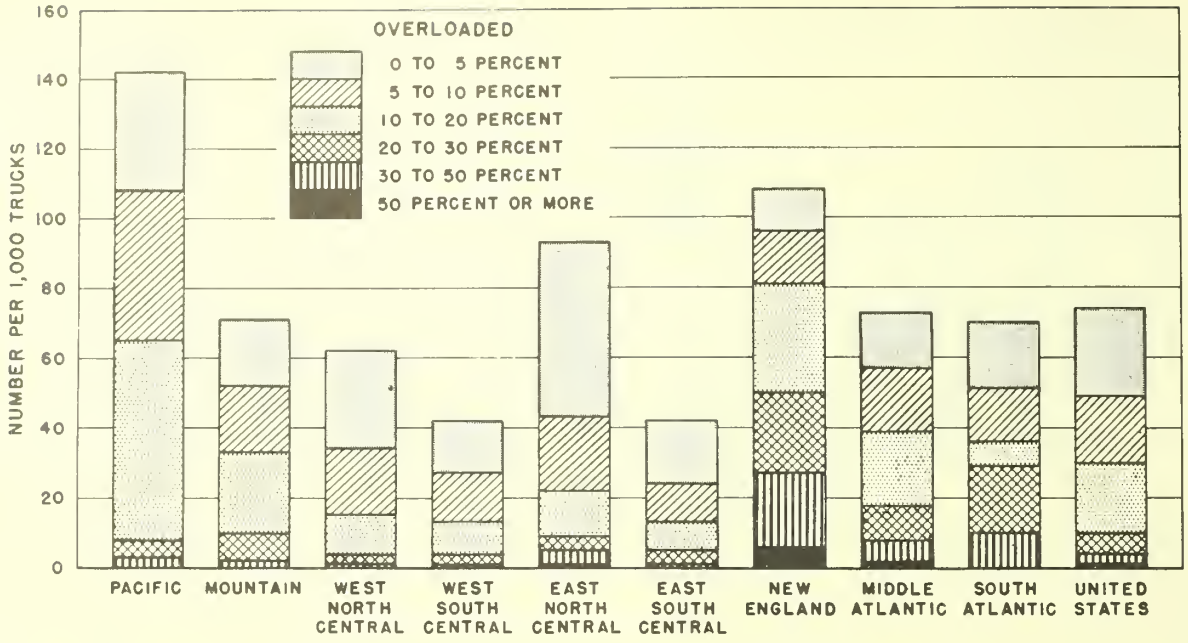

Figure 24.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the weight limits recommended by the AASHO, summer of 1954.

the States have approved higher load limits of several types, such as maximum axle loads of 22,400 pounds. Such an axle load is almost 25 percent heavier than the maximum recommended by the AASHO and would account for a considerable portion of the difference between the number of such vehicles exceeding State laws and those exceeding AASHO recommendations. Little overall trend in vehicles overloaded can be detected from the chart although some reduction in the number of those overloaded to a high degree can be noted, as for instance the frequency of overloads of 30 to 50 percent and of 50 percent or more.

The number of trucks and truck combinations in each 1,000 loaded and empty vehicles in the summer of 1954 that exceeded the axle, axle-group, or gross-weight limits in effect in the States or recommended by the AASHO are given in table 13. These latest frequencies when compared with similar figures for 1950 and 1953 are found to be well below them in practically every instance. However, the 1948, 1951, and 1952 frequencies are slightly below those for 1954.

The table is particularly interesting in that it shows that the major portion of overloads is confined to combination-type vehicles. Only one percent of the single-unit trucks exceeded any State weight limitation, while almost 14 percent of the combinations were overloaded to some extent. Of the combinations that involved a full trailer (in some instances the combination included a tractor, semitrailer, and a full trailer), more than 25 percent exceeded one of the axle, axle-group, or gross-load legal limits in effect in the States, and almost 44 percent exceeded to some extent the maximum weight recommendations of the American Association of State Highway Officials.

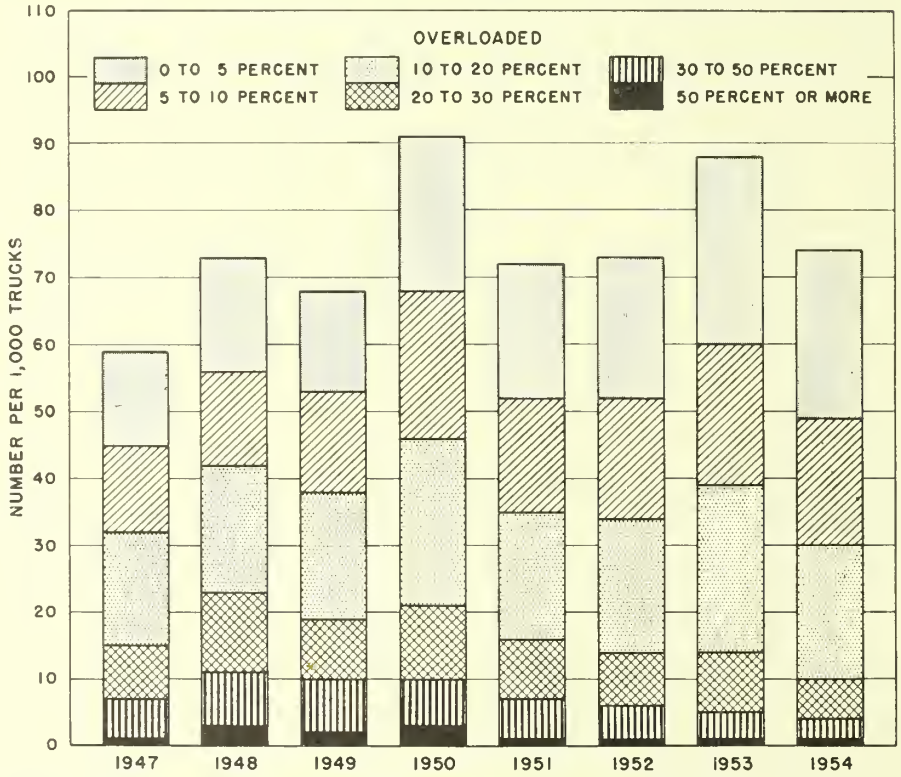

Figure 25.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the weight limits recommended by the AASHO, summers of 1947–54.

Table 13.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the axle, axle-group, or gross-weight limit in effect in the States or recommended by the AASHO in the summer of 1954 by main vehicle types and by regions

Region and type of vehicle	Vehicles exceeding State legal limits						Vehicles exceeding AASHO recommendations					
	Number per 1,000 overloaded	Number per 1,000 overloaded more than—					Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent		5 percent	10 percent	20 percent	30 percent	50 percent
New England:												
2-axle, 6-tire	10	6	4	2	1	1	60	50	44	32	19	10
3-axle	153	95	89	70	31		211	198	186	104	82	12
Average, single-unit trucks	12	7	6	4	2	(1)	39	34	30	20	13	6
Truck-tractor and semitrailer	123	84	51	15	5	1	302	268	223	134	66	7
Truck and trailer												
Average, truck combinations	123	84	51	15	5	1	302	268	223	134	66	7
Average, all trucks and combinations	41	28	18	7	3	1	108	96	81	50	27	6
Middle Atlantic:												
2-axle, 6-tire	21	14	5	1	1		52	45	31	15	5	1
3-axle	25	22	22	19	16	9	26	23	23	20	15	9
Average, single-unit trucks	13	8	4	1	1	(1)	30	26	18	9	3	1
Truck-tractor and semitrailer	74	41	25	8	3	(1)	144	107	72	33	15	4
Truck and trailer	77	77					85	81	81	77		
Average, truck combinations	74	41	25	8	3	(1)	144	106	72	33	15	4
Average, all trucks and combinations	36	21	12	4	2	(1)	73	57	39	18	8	2
South Atlantic:												
2-axle, 6-tire	18	10	5	3	1	(1)	28	18	11	6	2	(1)
3-axle	60	33	11	2			75	47	36	5	1	1
Average, single-unit trucks	11	6	3	1	1	(1)	15	10	6	3	1	(1)
Truck-tractor and semitrailer	159	90	57	19	5	1	199	147	105	42	14	1
Truck and trailer												
Average, truck combinations	158	89	56	19	5	1	198	146	104	42	14	1
Average, all trucks and combinations	55	31	19	6	2	(1)	70	51	36	29	10	2
East North Central:												
2-axle, 6-tire	7	5					8	6	(1)			
3-axle	21	13	7				51	23				
Average, single-unit trucks	4	3	(1)				6	3	(1)			
Truck-tractor and semitrailer	86	32	12	3	2	(1)	200	93	51	20	11	3
Truck and trailer	96	68					291	125	48	18		
Average, truck combinations	86	34	11	3	2	(1)	205	95	51	20	10	3
Average, all trucks and combinations	40	16	5	1	1	(1)	93	43	22	9	5	1
East South Central:												
2-axle, 6-tire	34	23	12	6	1	1	33	22	12	6	1	1
3-axle	32	14	6	5			32	14	6	5	4	
Average, single-unit trucks	16	10	6	3	1	(1)	16	10	6	3	1	(1)
Truck-tractor and semitrailer	158	91	49	13	3	(1)	142	79	43	11	2	(1)
Truck and trailer												
Average, truck combinations	158	91	49	13	3	(1)	141	79	43	11	2	(1)
Average, all trucks and combinations	45	27	15	5	1	(1)	42	24	13	5	1	(1)
West North Central:												
2-axle, 6-tire	9	4	2	1			9	4	2	1		
3-axle	42	21	10	4			15	5	4	4		
Average, single-unit trucks	6	3	1	(1)			5	2	1	(1)		
Truck-tractor and semitrailer	167	95	42	6	2		169	93	41	9	2	
Truck and trailer	136	66	58	57	57		129	64	58	57	57	
Average, truck combinations	166	93	43	8	4		167	92	42	11	4	
Average, all trucks and combinations	62	35	16	3	1		62	34	15	4	1	
West South Central:												
2-axle, 6-tire	16	11	6	1			16	10	5	1		
3-axle	54	45	9	9			54	45	9	9		
Average, single-unit trucks	6	4	2	1			6	4	2	(1)		
Truck-tractor and semitrailer	139	80	36	10	3	(1)	141	90	43	12	4	(1)
Truck and trailer	35	35	35				70	70	70	70		
Average, truck combinations	137	80	36	10	3	(1)	139	89	43	13	4	(1)
Average, all trucks and combinations	41	24	11	3	1	(1)	42	27	13	4	1	(1)
Mountain:												
2-axle, 6-tire	27	14	6	2	(1)		27	14	6	2	(1)	
3-axle	94	63	35	8	3		89	69	37	12	3	
Average, single-unit trucks	12	7	3	1	(1)		12	7	3	1	(1)	
Truck-tractor and semitrailer	207	136	88	25	6	1	235	172	111	37	10	1
Truck and trailer	256	158	103	16	6		333	260	174	30	6	
Average, truck combinations	217	140	91	24	6	1	255	189	124	35	9	1
Average, all trucks and combinations	62	39	25	7	2	(1)	71	52	33	10	2	(1)
Pacific:												
2-axle, 6-tire	34	25	12	(1)			34	26	12	(1)		
3-axle	134	47	20	7	(1)		148	76	20	13		
Average, single-unit trucks	18	10	5	(1)	(1)		19	12	5	1		
Truck-tractor and semitrailer	132	47	18	7	2		302	235	146	22	9	(1)
Truck and trailer	393	129	38	5	2		670	508	315	31	12	
Average, truck combinations	208	71	24	6	2		409	314	195	25	10	(1)
Average, all trucks and combinations	78	29	11	2	1		142	108	65	8	3	(1)
United States Average:												
2-axle, 6-tire	19	12	5	2	1	(1)	27	19	12	5	2	1
3-axle	65	34	18	9	3	1	74	47	27	12	6	2
Average, single-unit trucks	10	6	3	1	(1)	(1)	14	10	6	3	1	(1)
Truck-tractor and semitrailer	128	68	36	10	3	(1)	188	121	75	27	11	2
Truck and trailer	256	109	42	10	6		437	310	191	32	11	
Average, truck combinations	136	71	36	10	3	(1)	204	133	82	27	11	2
Average, all trucks and combinations	50	27	13	4	1	(1)	74	49	30	10	4	1

[1] Less than 5 per 10,000.

New Publications

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955 is the title of a new publication issued by the Bureau of Public Roads. The pamphlet contains a small-scale map of the United States showing the general location of the National System of Interstate Highways and individual maps showing the locations of routes of the system into, through, and around 102 urban areas. The urban maps show only the city outlines and the general locations of the Interstate system; detailed final locations will be determined as projects are advanced by the States for construction. The pamphlet contains no text other than the certificate of approval.

The publication is not distributed free by the Bureau. Copies may be purchased, at 55 cents a copy, from the Superintendent of Documents, U. S. Government Printing Office, Washington 25. D. C.

The National System of Interstate Highways, authorized by the Congress in 1944 with a limitation of 40,000 miles, is made up of the main highways of the Nation, connecting the principal metropolitan areas, cities, and industrial centers, and serving the national defense. It is a part of the larger Federal-aid primary system. The main network of the Interstate system, totaling 37,700 miles, was selected by joint action of the State highway departments and approved by the Bureau of Public Roads on August 2, 1947. The additional urban routes, totaling 2,300 miles, were recommended by the States and approved by the Bureau on September 15, 1955.

The urban maps in the pamphlet show without differentiation the general locations of both the newly added routes and the routes through the urban areas previously approved. Maps are not included for urban areas where new routes were not added. Many such urban areas are served by previously approved interstate routes passing through or around them, as indicated by the U. S. map of the system included in the pamphlet.

Opportunities in the Bureau of Public Roads for Young Engineers was recently published by the Bureau, in its third edition. This illustrated, 16-page pamphlet is intended to provide information for college students interested in careers in highway or highway bridge engineering in the Bureau of Public Roads. The publication briefly describes the organization, operations, and history of the Bureau, and explains in detail the 3-year, on-the-job training program offered by the Bureau to selected civil engineering school graduates.

The Bureau's free distribution of the pamphlet is restricted to use in recruitment for its training program. Others who are interested in the publication may purchase it from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

PUBLIC ROADS: World Traveler

PUBLIC ROADS magazine is not only one of the oldest continuously published periodicals of the United States Government; it is also probably one of the most widely traveled. A recent review of mailing lists shows that each bimonthly edition of more than 5,000 copies carries the message of the Bureau of Public Roads research work throughout the United States and to 80 countries around the world.

The Bureau of Public Roads distributes 817 copies of PUBLIC ROADS magazine to its own personnel, and 1,464 copies are sent to the State highway departments in furtheranee of the cooperative Federal-State highway program. In addition, 446 copies go to other Federal and State agencies and to county and local rural and urban government highway departments, 161 copies are sent to university libraries and instructors in highway engineering, and 138 to engineering and trade assoeiations and publications. The Bureau's free distribution overseas reaehes 50 foreign countries with 286 copies, mostly to national government highway ageneies.

In addition to the Bureau's distribution, the Government Printing Office, in its program of supplying representative libraries with Government publications, distributes PUBLIC ROADS to 373 depository libraries across the Nation. The United States Information Agency uses 87 copies in its information centers in 41 foreign countries.

Free distribution of the magazine by the Bureau of Public Roads is limited to public officials actually engaged in planning or constructing highways and to instructors of highway engineering. There are at present no vacancies on the free list, which is limited to a specific maximum by regulation. PUBLIC ROADS is sold by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1.00 per year (25 cents additional for foreign mailing). Orders for subscriptions are NOT received by the Bureau of Public Roads.

That PUBLIC ROADS magazine is considered worthwhile reading is indicated by the paid subscription list, numbering 1,018 in the United States and 523 in 60 foreign countries. In somewhat less than 2 years (December 1953 to October 1955), subseriptions in the United States have increased 29 percent and foreign paid subseriptions have practically doubled in number.

Geographically, the total (free and paid) PUBLIC ROADS distribution is remarkably widespread. It goes to every State, ranging in number from 15 copies for Vermont and 19 for Delaware to 315 for New York and 428 for California. Twenty-three States receive up to 50 copies eaeh; 13 States are in the 51 to 100 copy range; over 100 copies go to eaeh of 12 States and the District of Columbia. Four U. S. Territories are on the mailing lists.

In the Western Hemisphere, Canada and Mexico take 97 copies of the magazine; 8 countries in Central America and the Carribean area receive 22 copies; and 8 South American nations get 90 copies.

In Europe, 318 copies of the magazine are distributed among 22 different countries.

Dividing Africa on latitude 10° N., 6 North African countries receive 18 copies of PUBLIC ROADS; 10 Central and South African countries get 56 copies. On the Asian continent, 26 copies of the magazine go to 7 countries in the Middle East (longitude 30°–60° E.); 6 nations in Central Asia (longitude 60°–100° E.) receive 51 copies; 7 countries in the Far East (east of longitude 100° E.) get 126 copies. Four countries of Oceania receive 92 copies.

The 896 copies going to 80 different countries account for 17 percent of the total PUBLIC ROADS distribution of 5,313 copies.

The paid circulation of 1,541 copies amounts to 29 percent of the total distribution, which seems surprisingly good for a government periodical in the technical research field.

PUBLIC ROADS is indeed a world traveler and, judging from the proportion and rate of increase of subscription sales, it must be a welcome one.

As a matter that may be of interest to some, the countries receiving PUBLIC ROADS are listed below. The magazine also goes, of course, to all 48 States, the District of Columbia, Alaska, Canal Zone, Hawaii, and Puerto Rico.

Western Hemisphere.—Argentina, Bolivia, Brazil, British West Indies, Canada, Chile, Colombia, Costa Rica, Cuba, Ecuador, Guatemala, Honduras, Mexico, Nicaragua, Panama, Peru, Uruguay, and Venezuela.

Europe.—Austria, Belgium, Czechoslovakia, Denmark, England, Finland, France, Germany, Greeee, Hungary, Iceland, Ireland, Italy, Netherlands, Norway, Portugal, Scotland, Spain, Sweden, Switzerland, Union of Soviet Socialistic Republics, and Yugoslavia.

Africa.—Algeria, Angola, Belgian Congo, Egypt, Ethiopia, Federation of Rhodesia and Nyasaland, French West Africa, Gold Coast, Kenya, Liberia, Morocco, Mozambique, Nigeria, Tanganyika, Tunisia, and Union of South Africa.

Asia.—Afghanistan, Burma, Ceylon, China (Taiwan), Hong Kong, India, Iran, Iraq, Israel, Japan, Jordan, Lebanon, Malaya, Pakistan, Philippines, Singapore, Syria, Thailand, Turkey, and Kazakh Soviet Socialistic Republic (U. S. S. R.).

Oceania.—Australia, Indonesia, New Zealand, and Papua.

U. S. GOVERNMENT PRINTING OFFICE: 1955

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
1941, 15 cents. 1948, 20 cents.
1942, 10 cents. 1949, 25 cents.

Public Roads Administration Annual Reports:
1943; 1944; 1945; 1946; 1947.
(Free from Bureau of Public Roads)

Annual Reports of the Bureau of Public Roads:
1950, 25 cents. 1952, 25 cents. 1954 (out of print).
1951, 35 cents. 1953, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). 55 cents.

Construction of Private Driveways, No. 272MP (1937). 15 cents.

Criteria for Prestressed Concrete Bridges (1954). 15 cents.

Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.

Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.

Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.

Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Bridge Location No. 1486D (1927). 15 cents.

Highway Capacity Manual (1950). $1.00.

Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.

Highway Practice in the United States of America (1949). 75 cents.

Highway Statistics (annual):
1945 (out of print). 1948, 65 cents. 1951, 60 cents.
1946, 50 cents. 1949, 55 cents. 1952, 75 cents.
1947, 45 cents. 1950 (out of print). 1953, $1.00.

Highway Statistics, Summary to 1945. 40 cents.

Highways in the United States, *nontechnical* (1954). 20 cents.

Highways of History (1939). 25 cents.

Identification of Rock Types (1950). Out of print.

Interregional Highways, House Document No. 379 (1944). 75 cents.

Legal Aspects of Controlling Highway Access (1945). 15 cents.

Local Rural Road Problem (1950). 20 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways (1948) *(including 1954 revisions supplement)*. $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

Mathematical Theory of Vibration in Suspension Bridges (1950) $1.25.

Model Traffic Ordinance (revised 1953). 20 cents.

PUBLICATIONS (Continued)

Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.

Opportunities in the Bureau of Public Roads for Young Engineers (1955). 25 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.

Public Control of Highway Access and Roadside Development (1947). 35 cents.

Public Land Acquisition for Highway Purposes (1943). 10 cents.

Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

Roadside Improvement, No. 191MP (1934). 10 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.

Standard Plans for Highway Bridge Superstructures (1953). $1.25.

Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.

Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.

United States System of Numbered Highways 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24–27.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1955

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$19,913	$13,218	$6,731	324.0	$6,035	$3,339	57.0	$38,210	$19,646	597.8	$57,463	$29,716	978.8
Arizona	8,961	4,884	3,616	93.9	1,565	1,176	29.7	11,048	8,071	184.4	17,497	12,863	308.0
Arkansas	15,227	9,565	5,129	315.4	7,121	3,564	82.9	19,525	9,866	433.6	36,211	18,559	831.9
California	18,293	43,690	23,703	222.9	15,081	8,160	27.4	129,735	66,046	286.9	188,506	97,909	537.2
Colorado	18,123	10,353	5,864	165.8	2,448	1,417	20.1	17,676	9,595	146.0	30,477	16,876	331.9
Connecticut	23,204	840	420	3.7	757	374	3.2	8,135	4,045	11.8	9,732	4,839	18.7
Delaware	6,850	1,723	868	10.2	1,207	621	20.5	5,911	3,299	14.8	8,841	4,788	45.5
Florida	17,279	14,196	7,453	288.3	12,815	6,611	83.6	25,520	12,808	236.9	52,531	26,872	608.8
Georgia	26,351	24,616	12,417	527.2	7,622	3,399	64.0	39,780	18,862	682.6	72,018	34,678	1,273.8
Idaho	10,039	3,857	2,488	77.4	4,025	2,686	42.6	9,547	6,194	152.4	17,429	11,368	272.4
Illinois	30,895	39,413	22,567	378.9	20,056	11,049	43.2	86,210	45,717	575.7	145,679	79,333	996.9
Indiana	33,164	18,745	9,561	90.7	13,346	7,716	54.0	37,239	19,890	100.2	69,330	37,167	244.9
Iowa	18,375	17,730	9,630	513.5	5,831	2,974	83.0	23,471	12,669	889.1	47,032	25,273	1,485.6
Kansas	19,078	10,518	5,262	657.5	2,613	1,400	65.7	22,284	11,339	918.9	35,415	18,001	1,642.1
Kentucky	15,776	11,513	6,292	100.2	3,197	1,932	10.4	40,337	20,720	635.0	55,047	28,950	745.6
Louisiana	13,648	15,019	7,509	169.7	5,451	2,725	32.3	42,974	20,368	510.5	63,444	30,602	712.5
Maine	8,018	9,198	4,695	65.0	543	272	5.9	11,385	5,878	76.0	21,126	11,045	146.9
Maryland	9,755	28,618	14,540	72.9	5,865	2,821	4.8	14,069	7,362	86.6	48,552	24,723	164.3
Massachusetts	27,423	8,354	4,167	24.9	1,806	903		43,725	20,751	46.9	53,885	25,821	71.8
Michigan	20,031	48,882	25,674	534.4	18,783	9,790	71.2	47,124	23,742	545.6	114,789	59,206	1,151.2
Minnesota	23,207	5,661	2,962	332.5	3,282	1,656	128.3	31,147	16,606	666.7	40,090	21,224	1,127.5
Mississippi	10,556	7,313	3,591	249.0	4,119	2,243	62.2	28,512	14,686	740.8	39,944	20,520	1,052.0
Missouri	18,378	25,810	13,233	1,123.0	8,783	5,280	8.0	72,387	37,593	1,156.8	106,980	56,106	2,287.8
Montana	10,699	9,610	6,035	187.7	5,594	3,235	90.5	21,918	13,766	370.0	37,122	23,036	648.2
Nebraska	17,295	18,322	9,572	841.4	6,404	2,919	59.2	27,252	15,241	770.0	51,978	27,732	1,670.6
Nevada	14,811	3,910	3,307	60.6	666	557	13.3	5,140	4,316	92.1	9,716	8,180	166.0
New Hampshire	7,421	2,225	1,120	14.6	158	79	.8	7,190	3,725	44.5	9,579	4,924	59.9
New Jersey	25,817	13,356	6,146	57.5	3,556	1,786	4.7	30,447	14,209	53.0	47,359	22,141	115.2
New Mexico	11,629	3,584	2,317	54.0	2,795	1,827	39.4	8,696	5,759	170.9	15,075	9,903	264.6
New York	66,564	48,717	25,196	71.4	8,556	4,375	21.6	228,815	105,411	333.8	286,088	134,982	426.8
North Carolina	24,428	15,685	7,880	187.3	4,108	1,979	47.9	41,729	20,803	590.3	61,662	30,662	825.5
North Dakota	12,270	5,224	2,689	805.3	4,260	2,168	435.1	11,137	5,597	651.6	20,621	10,454	1,895.0
Ohio	36,682	56,819	29,284	171.1	7,053	3,447	25.5	66,129	31,933	103.4	130,001	64,664	300.0
Oklahoma	19,574	17,111	8,940	337.5	14,301	7,541	141.2	30,342	15,891	359.7	61,754	32,372	838.4
Oregon	8,056	11,671	6,708	108.0	4,366	2,761	42.7	16,175	10,029	159.0	32,212	19,498	309.7
Pennsylvania	40,014	56,559	28,334	76.7	20,779	10,761	44.0	82,960	41,545	268.2	160,309	80,640	388.9
Rhode Island	2,265	10,189	5,414	18.6	946	473	1.5	14,076	7,051	29.1	25,211	12,938	49.2
South Carolina	16,427	11,912	6,390	329.6	1,911	1,163	18.3	20,707	10,759	317.5	34,530	18,312	665.4
South Dakota	12,978	8,850	5,097	382.4	3,534	2,057	124.4	11,316	6,523	536.5	23,700	13,677	1,043.3
Tennessee	23,999	16,109	9,013	247.2	8,087	4,043	88.8	37,310	16,870	358.5	63,506	29,926	694.5
Texas	44,982	30,077	15,867	550.4	18,269	9,709	167.3	88,662	46,954	1,262.5	137,008	72,530	1,980.2
Utah	7,570	5,798	4,329	74.0	1,698	1,181	5.1	11,570	8,833	195.1	19,066	14,343	274.2
Vermont	5,632	2,025	1,019	8.9	263	132	7.0	8,965	4,699	86.5	11,253	5,850	102.4
Virginia	21,038	15,856	8,222	256.7	4,536	2,374	25.9	24,801	12,066	281.9	45,193	22,662	564.5
Washington	14,291	15,399	8,475	154.5	3,863	2,019	72.2	23,859	12,733	125.8	43,121	23,227	352.5
West Virginia	18,402	8,891	4,547	37.3	2,757	1,393	7.8	17,575	8,824	98.6	29,223	14,764	143.7
Wisconsin	22,776	12,575	6,454	125.2	2,339	1,176	7.1	47,568	23,892	487.0	62,482	31,522	619.3
Wyoming	8,284	2,474	1,593	33.1	1,242	799	18.4	13,018	8,547	273.5	16,734	10,939	325.0
Hawaii	5,329	2,827	1,414	5.7	3,177	1,571	2.9	5,050	2,245	11.5	11,054	5,230	20.1
District of Columbia	10,130	4,437	2,218	4.1	1,206	602	2.4	8,175	3,616	1.3	13,818	6,436	7.8
Puerto Rico	12,817	3,443	1,668	21.0				16,591	7,602	57.6	20,034	9,270	78.6
TOTAL	652,024	789,502	417,826	11,564.9	288,775	154,235	2,515.3	1,733,150	885,192	17,785.4	2,811,427	1,457,253	31,865.6

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Evening peak-hour traffic on 13th Street, NW., Washington, D. C.

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

Contents of this publication may be reprinted. Mention of source is requested.

Capacities of One-Way and Two-Way Streets with Signals and with Stop Signs

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH [1]
Highway Transport Research Engineer

In recent years there have been reports from several cities of streets with progressive signal control carrying very heavy traffic at comparatively high speed. Attempts to develop comparable systems in other areas have frequently been unsuccessful. This article describes an intensive study of a major street and indicates the high capacities which are attainable. With information from a number of these facilities it should be possible to determine and measure the effects of the significant factors which make such high capacities possible.

The possible capacity of the one-way through street, 40 feet in width, described in this article was found to be 7,000 vehicles during each hour of green signal time. With the light green during one-half of each signal cycle, it is thus possible to carry 3,500 vehicles during each clock hour. Since an ideal combination of conditions, some of which remain to be measured, is evidently necessary for the attainment of these high volumes, there is no assurance that such high capacities can be designed into a city street system. The traffic volumes given in the Highway Capacity Manual are usually attainable under average conditions, and the values reported therein are generally appropriate for design purposes.[2]

All-way stop signs, the general term which includes four-way stop signs as well as the situation when one or more of the streets are one way, have been used frequently during recent years. There has been considerable discussion by engineers as well as the driving public of the relative merits of all-way stop signs, the more usual stop signs on the cross streets only, and signal control. In this article capacities with signal control, with stop signs on the cross street, and with stop signs on all approaches are compared. From this comparison it appears that stop signs on the cross streets only are practical at traffic volumes considerably below those at which the other two types of control are feasible. When both streets are two way, four-way stop signs appear to be practical at traffic volumes approaching those observed for signal control. With the through street one way, however, progressive signal control was found to provide substantially greater capacity than either of the two types of stop-sign control.

[2] *Highway capacity manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board. Published by the Bureau of Public Roads, 1950.

THE relative merits of each of the various types of control devices have long been a topic of discussion and it is therefore desirable that factual information be obtained for use as a guide in determining the conditions under which traffic signals, two-way stop signs, and four-way stop signs provide the most efficient operation.

Both stop signs and signals are used extensively to control traffic at intersections. Generally, stop-sign control is used only on the less important of the two intersecting streets, but in some instances four-way stop-sign control is used.

This article is concerned with a study of traffic at four intersections while the traffic signals were in normal operation, and also studies at the same locations while two-way stop signs and four-way stop signs were used in place of traffic signals.

Definitions

Progressive signal control.—In the simple progressive system the various signal faces controlling a given street give "go" indications in accordance with a time schedule to permit, as nearly as possible, continuous operation of groups of vehicles along the street at a planned rate of speed, which may vary in different parts of the system.

Possible capacity.—The maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the prevailing roadway and traffic conditions, regardless of their effect in delaying drivers and restricting their freedom to maneuver.

Practical capacity.—The maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the prevailing roadway and traffic conditions, without unreasonable delay or restriction to the drivers' freedom to maneuver.

Cross street stop-sign control.—The cross street approach or approaches to an intersection are controlled by stop signs while the through street is uncontrolled.

All-way stop-sign control.—All intersection approaches are controlled by stop signs. The term "four-way stop sign" is not applicable at intersections where one or both of the intersecting streets are one way.

Loaded cycles.—A cycle was considered to be loaded at an intersection approach if vehicles were traveling through the intersection during the entire green period and the flow was stopped by the amber or red signal.

Description of Study Site

The four intersections studied are on 13th Street, N. W., in Washington, D. C., a four-lane street which carries more than 3,200 vehicles per hour during the peak period. It is operated one way inbound during the morning peak period and one way outbound during the evening peak period, and for the remaining time it operates as a two-way street with parking on one side. The 1.8-mile section of 13th Street between Logan Circle and Spring Road with a peak-hour overall speed of about 18 miles per hour is an unusually efficient arterial street, carrying far heavier traffic than

[1] The data were collected in the field by 19 junior highway engineers. A preliminary report by four of these engineers, Robert D. Bee, Walter W. Bryant, Dwight A. Hodgens, Jr., and Joseph Rekas, was prepared as part of the Bureau of Public Roads Junior Engineer Training Program. This article is based chiefly on further analysis of the data. The District of Columbia Department of Motor Vehicles and Traffic supplied the signs and adjusted signal timing as directed by John H. Mitton, Assistant Director and Traffic Engineer. The Metropolitan Police provided officers for emergency control under the direction of John J. Agnew, Deputy Chief of Police.

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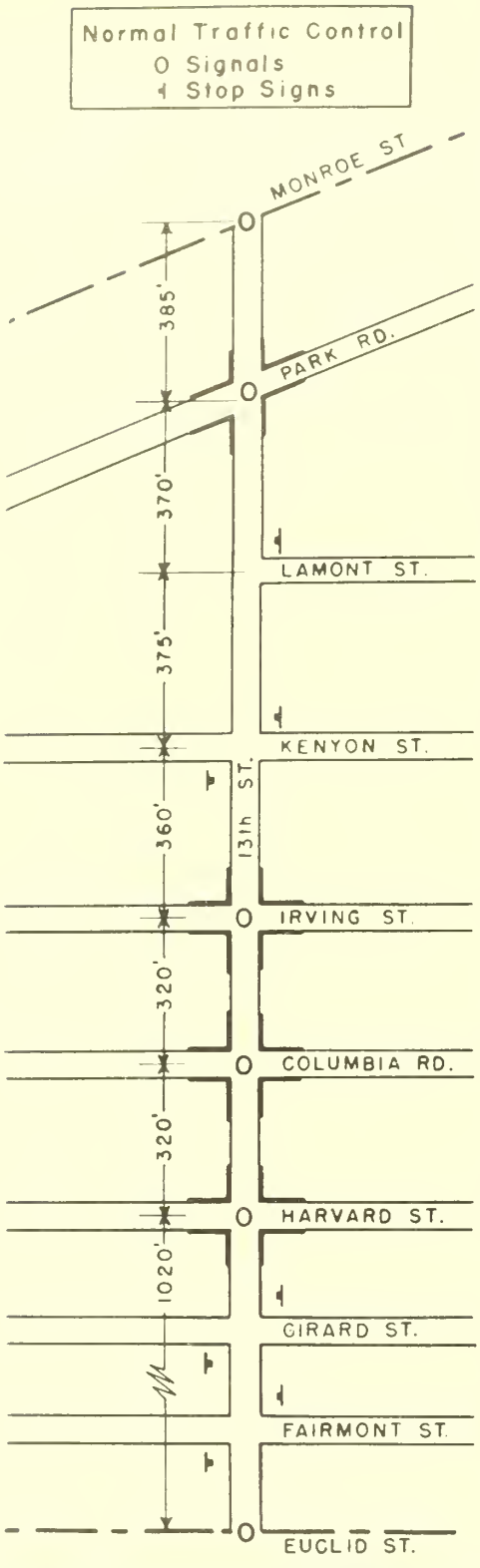

Figure 1.—Location of four intersections on 13th Street included in study.

most streets of this width are capable of accommodating under usual conditions. The three types of intersection control were observed under heavy traffic loads at intersections with both one-way and two-way streets.

Thirteenth Street is a through north-south street carrying much heavier traffic volumes than the cross streets. The cross streets at which studies were made are as follows: Irving Street and Park Road, which carry two-way traffic; Harvard Street, which is one way eastbound; and Columbia Road, which is one way westbound. The general layout of the study area is shown in figure 1. Details of each intersection are shown in figure 2, while figure 3 shows typical traffic on 13th Street when operating two way and one way.

All four intersections are normally controlled by an interconnected system of coordinated fixed-time traffic signals. There is a single dial in the time controller at each intersection. Consequently, the 80-second signal cycle and the stop and go intervals for 13th Street and the cross streets remain constant throughout the day. The signals are interconnected with a master controller, however, so that the time offsets between successive signals on 13th Street can be changed. This provides for progressive traffic movement favoring the desired direction of travel at different times of the day.

Three different signal progressions are used. The first, which is used during the morning rush period, provides progressive movement for 13th Street traffic while it is one way inbound. The second, which is used during nonrush periods, provides reasonably good progression in both directions while 13th Street carries two-way traffic. The third, which is used during the afternoon rush period, provides excellent progression while 13th Street is operating one way outbound.

Accident data for 13th Street are compared with those for other arterial streets and urban freeways in table 1. The accident rate is 13 percent higher than the average based on a nationwide study of representative arterial city streets. This higher accident rate may be due in part to the very complete reporting of accident data in the District of Columbia. It is not believed to indicate a significantly higher accident potential. The accident rate on 13th Street is much higher, however, than that for urban freeways with full control of access although the fatality rates are the same.

Scope of Study

The principal field studies were conducted on three weekday afternoons in March 1954. Operation with signal control was observed the first day. The green time on the cross streets was reduced in an attempt to provide a continual backlog of vehicles on cross streets so that their possible capacities could be determined. On another day, stop signs were used to control traffic on the cross streets. On the third day, stop-sign control was used on all approaches to the intersections. Additional field studies were conducted in November to measure the delay to traffic with the normal signal timing and to determine the capacity of 13th Street with two-way traffic controlled by signals.

Manual counts of all through and turning movements, classified by type of vehicle, were made at all entrances to the intersections. These were recorded for every cycle when signals were in operation, and every 2 minutes when signals were off and traffic was being controlled by stop signs. Fully utilized "go" periods (loaded cycles) were noted while observing traffic controlled by signals. To provide data from which vehicular delay might be computed, counts were made of standing vehicles at each intersection approach at regularly spaced intervals. Intervals of 30 seconds were used when signals were in operation and 2-minute intervals were used when stop-sign control was employed.

Field data were summarized by 10-minute periods. Rates per hour of green signal time were calculated so that the rates are comparable regardless of signal timing. The rate per hour of green was calculated by dividing the hourly volume as normally determined by the percentage that the green period, not including any amber time, was of the total cycle.

To calculate total delay, the number of stopped vehicles as determined by the periodic count was multiplied by the time interval between counts. This value was then divided by the number of vehicles entering the intersection to determine the delay per vehicle.

Table 1.—Accidents per hundred million vehicle-miles of travel on 13th Street compared with other facilities

	13th Street [1]	Other arterial streets [2]	Urban freeways [2]
Reported accidents	1,091	966	146
Fatalities	2	3	2

[1] Logan Circle to Spring Road, 1.8 miles, Jan. 1, 1952-June 30, 1954.

[2] These are preliminary figures for arterial streets with no access control and for urban freeways with full control of access. Data were developed from cooperative studies made by the Bureau of Public Roads and State highway departments.

Summary of Findings

A definite value for either practical or possible capacity was determined for each of the three types of intersection control included in this study. The effect of one-way and two-way operation on the capacities of both through and cross streets and the effect of parking on the through street were also determined. These determinations were based on the magnitude of the delays to traffic as well as on the traffic volumes observed. The studies were conducted at intersections where the through street is 40 feet wide and the cross streets 30 feet wide with parking on one side. The intersections are in a densely developed residential area with comparatively few pedestrians. Traffic on the through street was exceedingly heavy during the rush periods, and on each of the four important cross streets it was seldom less than 250 vehicles per hour during the day. The following are the more important findings for the prevailing roadway and traffic conditions:

Progressive signal control

The capacity of 13th Street with one-way operation and parking prohibited was found to be 1,750 vehicles per hour of green per 10 feet of street width.

The capacity of 13th Street when operating two way with parking on one side was 37

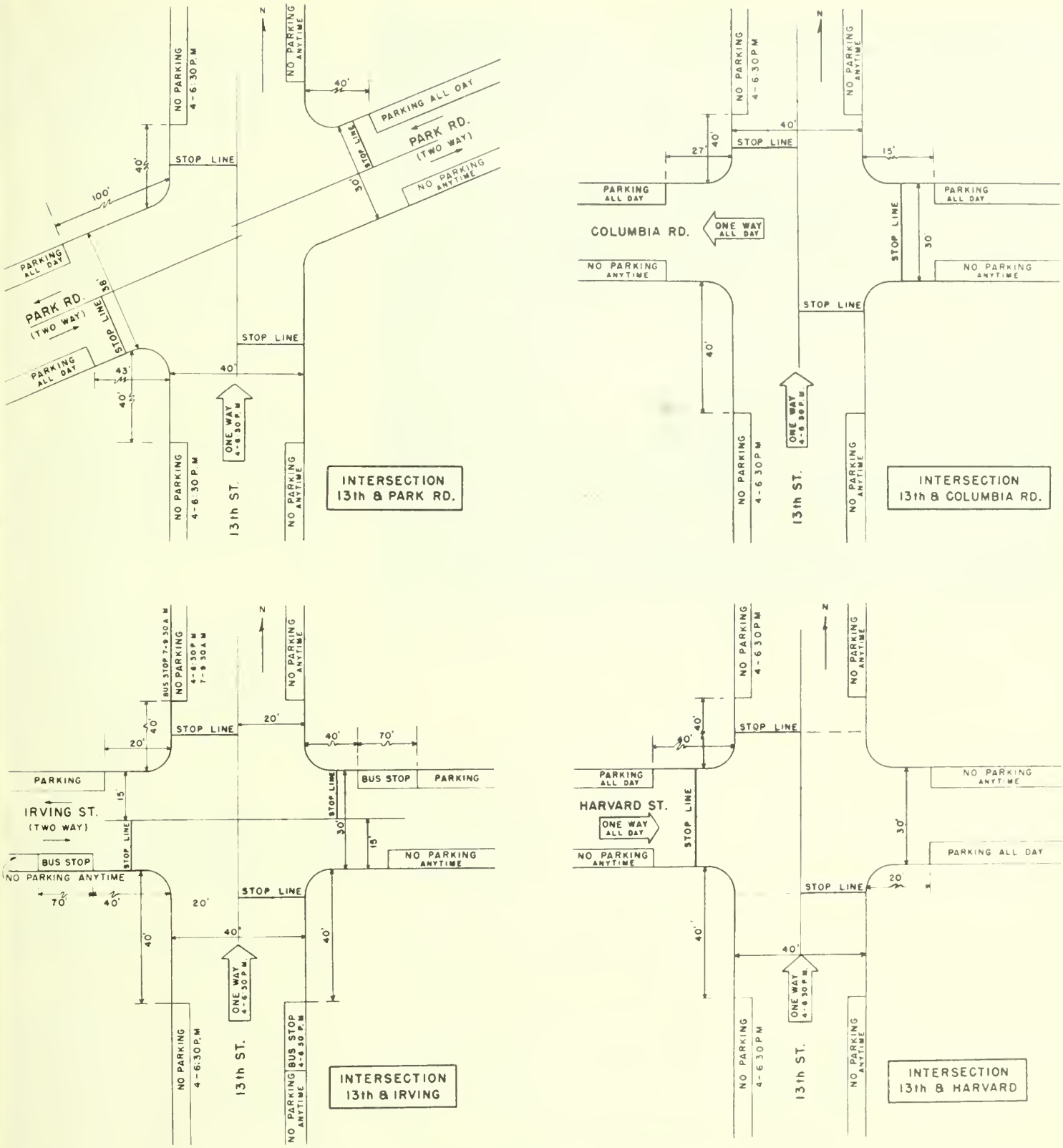

Figure 2.—Detailed information for intersections studied.

percent greater at two-way cross streets and 48 percent greater at one-way cross streets than with parking on both sides. With no parking on either side, the through-street capacity was 75 percent greater at two-way cross streets and 97 percent greater at one-way cross streets than when parking was permitted on both sides.

Changing from two-way to one-way operation on the through street, with no parking, increased the capacity of the through street 25 percent at intersections with two-way cross streets and 11 percent at intersections with one-way cross streets.

The capacity of the through street was increased 119 percent by the elimination of parking from both sides and by the use of one-way rather than two-way operation.

Changing from two-way to one-way operation and eliminating parking on the through street increased the capacity of the two-way cross streets 11 percent and the one-way cross streets 20 percent.

The capacity of the one-way cross streets

Figure 3.—Traffic scenes on 13th Street: two-way traffic (above) during off-peak period at Irving Street intersection, and one-way northbound traffic (below) during evening peak period at Park Road intersection.

was 15 percent greater than the capacity of the two-way cross streets while there was two-way operation on 13th Street. The corresponding figure while 13th Street was operating as a one-way street was 24 percent.

Traffic delays with progressive signal control increased comparatively little with an increase in the traffic volume until the volumes approached very closely the possible capacities of the streets. This was true for delays to both through- and cross-street traffic.

Stop-sign control on cross streets

The capacities of the cross streets with stop-sign control on these streets only were affected principally by the traffic volume on the through street. Whether the cross streets or the through street were operated as one-way or two-way streets made no apparent difference in the capacity of the cross street. The possible capacity of one-way and two-way cross streets decreased from 800 vehicles per hour to 400 vehicles per hour when the volume on the through street increased from 1,300 to 2,500 vehicles per hour. The decrease in capacity of the cross streets was not, however, directly proportional to the increase in volume on the through street.

Delay to traffic on the through street was practically nil, regardless of the traffic volume on the cross streets.

The delay to traffic on the cross streets was the principal criterion for determining possible and practical capacities with stop signs on the cross streets. The possible capacity is the volume which, if exceeded even a slight amount, will result in extremely long delays. For a given volume of traffic on the through street, the delay to traffic on the cross street when operating at possible capacity is more than double the delay when operating at practical capacity. Also, the practical capacity of a cross street is less than one-half of its possible capacity.

Changing either the through street or the cross street from two-way to one-way operation had no apparent effect on the delay to traffic on the cross street or the capacity of the cross street.

All-way stop-sign control

The practical capacity of the intersections with all-way stop-sign control was found to approach 500 vehicles per lane per hour for the operational lanes on both streets. The validity of this finding under other conditions with a high traffic volume per lane on one of the streets and a low volume per lane on the other street was not determined.

With similar traffic volumes per lane on both streets, the delay per vehicle on the through street was approximately the same as the delay per vehicle on the cross street.

Conclusions

The following conclusions appear to be justified for the conditions under which these studies were conducted:

1. With properly coordinated progressive signal control, the practical capacity of the one-way street closely approaches its possible capacity. The delays to traffic when volumes are near possible capacity are not excessive under these conditions and only slightly greater than the delays at much lower volumes.

2. Intersection capacities are greater with progressive signal control than with either type of stop-sign control. When the through street is one way, the intersection capacities with progressive signal control are substantially greater than with either type of stop-sign control. The average delay to all traffic is less with progressive signal control than with either type of stop-sign control, except possibly with stop-sign control at cross streets carrying exceedingly low volumes while the through street is carrying a high volume.

3. Possible and practical capacities for cross-street stop-sign control are much lower than for progressive signal control or all-way stop-sign control. Somewhat higher volumes than those found during this study might be practical with stop-sign control on the cross street when the intersection is located between signal-controlled intersections on the through street.

4. The capacity of an intersection at which all traffic is controlled by stop signs approaches that of an intersection with progressive signal control when both streets carry two-way traffic. When one or both of the streets carry traffic in only one direction, the capacity with all-way stop signs is considerably lower than that possible with progressive signal control.

5. The capacity of an intersection will vary greatly with the control and regulation of traffic. A change in operating conditions at one intersection approach may affect the capacity of the intersecting street as well as that of the street on which the change occurred. Additional studies are necessary to determine the most effective type of control for the many conditions that exist other than those included in this study.

In the application of the above conclusions, it must be remembered that they apply only to conditions similar to those found on 13th Street. The very high values for capacity with signal control cannot be used when estimating capacities because for most streets the results would be erroneously high. The very efficient operation of the progressive signal system is emphasized since it expedites the movement of exceedingly high volumes of traffic to a degree seldom equaled on streets of this type. It may be significant that most of the drivers on 13th Street during the peak period use this street daily and are therefore practiced in maintaining optimum speed and spacing.

These facts are important considerations when comparing the results of this study with those of other studies which do not show the same advantages for signal control as com-

pared with stop-sign control.[3] There is close agreement, however, in the results of this study and other studies in the traffic volumes accommodated by the two types of stop-sign control. The fact that 13th Street can accommodate smoothly such high traffic volumes at a reasonable speed for urban conditions is a tribute to the traffic engineers who operate the progressive signal system.

Signal Control

Despite the high traffic volumes observed at the four intersections while they were controlled by signals, the traffic demand on some of the approaches was insufficient to utilize fully the green or "go" interval during any of the signal cycles. On several of the approaches the green interval was fully utilized during a few cycles only. The green interval was considered fully utilized when the traffic demand at the observed approach was equal to the possible capacity of that intersection approach with vehicles continuously entering the intersection throughout the green interval. Cycles during which these conditions occurred are called loaded cycles. A loaded cycle for an intersection approach is independent of the traffic on the other approaches which may or may not be loaded during the same period.

The first four columns of table 2 show the traffic volume on 13th Street, the percentage of dual-tired vehicles, and the percentage of right and left turns during the 1-hour period of maximum traffic. The next column shows the number of seconds of green signal time per cycle. The traffic volume per hour of green signal time follows. The highest hourly rate observed during 10 consecutive minutes is shown next, followed by the rate per hour of green time for loaded cycles. All these data were compared for similar intersection approaches, and the estimated possible capacity was determined for each approach. This is tabulated in the last column of the table. Table 3 is similar to table 2 and lists the data for the cross streets.

Determination of possible capacity

The estimated possible capacities shown in the last column of tables 2 and 3 were determined on the basis of the 1-hour, 10-minute, and loaded-cycle volumes. Data for similar intersection approaches were considered collectively in arriving at these capacities. The values represent the possible capacities over a 1-hour period even though volumes for shorter time periods were used in their determination. A detailed description of the method of determining these values is necessary for an appreciation of their reliability.

First, the 1-hour volume was compared with the maximum 10-minute volume and the volume during loaded cycles at the particular approach. The relation between these values as well as the frequency of loaded cycles formed a basis for tentatively estimating possible capacity. For instance, if the 1-hour volume and loaded-cycle volume were well below the maximum 10-minute volume, it was evident that the first two were well below capacity. The presence of only a few loaded cycles during a large number of consecutive cycles would confirm this. In such a case it is possible that even the maximum 10-minute volume was below possible capacity. On the other hand, if the three volumes were about equal and a large and fairly concentrated number of loaded cycles were observed, this was an indication that possible capacity was reached. In such instances a volume somewhat below the maximum 10-minute volume, but not less than the loaded cycle volume, might be tentatively selected as the possible capacity.

After the tentative values for all similar

Table 2.—Traffic data and estimated possible capacity for 13th Street at four signal-controlled intersections

Location	Conditions during continuous 1-hour periods of maximum traffic					Maximum observed traffic volumes per hour of green time during—			Estimated possible capacity per hour of green
	Traffic volume	Dual-tired vehicles	Turning movements		Green time for 80-second cycle	1-hour period	10-minute period	Loaded cycles [1]	
			Left	Right					
13TH STREET CARRYING TWO-WAY TRAFFIC [2]									
13th Street, southbound:	*V. p. h.*	*Percent*	*Percent*	*Percent*	*Seconds*	*Vehicles*	*Vehicles*	*Vehicles*	*Vehicles*
At Park Rd., two way	482	3.5	0.4	7.0	38	1,015	2,080	1,530 (1–1)	1,600
At Irving St., two way	524	2.9	1.9	1.3	[3]	1,435	1,720	1,605 (25–39)	1,600
At Columbia Rd., one way	557	3.4	.....	6.8	45	990	1,181	None	1,600
At Harvard St., one way	555	4.3	5.4	.....	29	1,531	1,707	1,610 (55–78)	1,600
13th Street, northbound:									
At Park Rd., two way	836	2.7	4.1	6.6	40	1,672	2,124	2,460 (3–3)	2,600
At Irving St., two way	748	2.9	4.3	1.3	[4]	2,109	2,916	2,635 (10–25)	3,000
At Columbia Rd., one way	671	3.3	6.8	.....	45	1,193	1,611	None	3,000
At Harvard St., one way	849	2.8	.....	4.1	29	2,342	3,321	3,576 (5–20)	3,300
13TH STREET CARRYING ONE-WAY TRAFFIC [2]									
13th Street, northbound:									
At Park Rd., two way	2,747	0.9	6.4	10.3	40	5,494	7,140	6,857 (8–16)	7,000
At Irving St., two way	3,128	1.0	1.8	.5	52	4,811	5,623	None	7,000
At Columbia Rd., one way	3,168	1.0	3.6	.....	45	5,633	6,614	None	7,000
At Harvard St., one way	2,987	.8	.....	4.7	50	4,779	5,309	5,637 (4–16)	7,000

[1] The number of loaded cycles observed is shown by the first of the two figures in parentheses; the second figure is the number of consecutive cycles in the period during which the loaded cycles were observed.

[2] Parking is permitted only on the west side used by southbound traffic while 13th Street is operating two way. All parking is prohibited on 13th Street during one way operation.

[3] Twenty-four, 32-second green periods; eleven, 28-second green periods; and ten, 24-second green periods.

Table 3.—Traffic data and estimated possible capacity of four signal-controlled streets crossing 13th Street [1]

Location	Conditions during continuous 1-hour periods of maximum traffic					Maximum observed traffic volumes per hour of green time during—			Estimated possible capacity per hour of green
	Traffic volume	Dual-tired vehicles	Turning movements		Green time for 80-second cycle	1-hour period	10-minute period	Loaded cycles [2]	
			Left	Right					
CROSS STREETS CARRYING TWO-WAY TRAFFIC									
13th Street, two way:	*V. p. h.*	*Percent*	*Percent*	*Percent*	*Seconds*	*Vehicles*	*Vehicles*	*Vehicles*	*Vehicles*
Park Rd., eastbound	273	8.0	2.9	9.9	18	1,213	1,502	1,333 (24–40)	1,400
Park Rd., westbound	294	3.4	25.8	1.7	18	1,307	1,551	1,400 (30–45)	1,400
Irving St., eastbound	203	13.8	13.8	18.7	18	902	1,200	1,120 (10–44)	1,200
Irving St., westbound	115	15.6	6.1	7.8	18	511	649	800 (1 1)	1,200
13th Street, one way:									
Park Rd., eastbound	366	2.7	13.1	.....	18	1,627	1,849	1,542 (43–52)	1,600
Park Rd., westbound	304	3.0	.....	4.9	24	1,013	1,249	1,433 (6–52)	1,400
Irving St., eastbound	245	9.8	19.6	.....	18	1,089	1,450	1,055 (40–92)	1,400
Irving St., westbound	179	7.3	.....	16.8	18	795	1,000	850 (8–39)	1,400
CROSS STREETS CARRYING ONE-WAY TRAFFIC									
13th Street, two way:									
Columbia Rd	465	3.9	7.3	9.0	15	2,480	2,917	2,824 (13–28)	3,000
Harvard St	447	5.6	20.4	5.1	15	2,384	2,821	3,120 (5–7)	3,000
13th Street, one way:									
Columbia Rd	668	1.0	.....	12.6	[3]	3,185	3,960	3,553 (41–95)	3,600
Harvard St	708	1.8	33.5	.....	17	3,332	3,936	3,600 (60–89)	3,600

[1] Cross streets are 30 feet wide with parking permitted on one side except for the west leg of the Park Road intersection. The latter is 38 feet wide with parking permitted on both sides.

[2] The number of loaded cycles observed is shown by the first of the two figures in parentheses; the second figure is the number of consecutive cycles in the period during which the loaded cycles were observed.

[3] Thirty-seven, 15-second green periods; and eight, 25-second green periods.

[3] *A capacity relationship between four-way stop intersection control and fixed-time traffic signal*, by James Madison Hunnicutt, Jr. Thesis, Bureau of Highway Traffic, Yale University, 1954; also *A comparison of delay to vehicles crossing urban intersections, four-way-stop vs. semi-traffic-actuated signal control*, by Edward M. Hall. Student Research Report No. 4, The Institute of Transportation and Traffic Engineering, University of California, Jan. 1952.

intersection approaches were selected by this process they were compared with each other for consistency and reasonableness. In this comparison differences in turning movements, frequency of commercial vehicles, bus stops, and other factors known to affect traffic operation were considered. If the tentative possible capacity estimate for any approach appeared inconsistent, a reappraisal was made. In the case of similar approaches where the tentative estimates were close, a single value was selected for the possible capacity of all. If only one of several otherwise similar approaches was observed at or near possible capacity conditions, the value determined for this approach was assigned to the others.

Unusually high values were determined for the possible capacity of 13th Street when it was operating one way northbound. The highest volumes per hour of green were observed at Park Road where the green time was least. Even here, with only one-half the total cycle green to 13th Street, only eight loaded cycles occurred. With a volume during these cycles of 6,857 vehicles per hour of green and a maximum 10-minute volume of 7,140 vehicles per hour of green, the indicated 7,000 vehicles per hour of green is evidently the approximate possible capacity. Since the turning movements were less at the other approaches, it must be inferred that possible capacities at these approaches are at least equal to the 7,000 vehicles per hour of green determined at Park Road. In the absence of factual data indicating higher capacity, this value was accepted for all four locations, rather than some higher value.

The results of an analysis of the data in tables 2 and 3 indicate that within the range of values observed, the percentage of commercial vehicles and the percentage of left and right turning traffic at the intersections had little apparent effect on capacities of the approaches. An exception is the case of Irving Street which carried about twice as high a percentage of dual-tired vehicles as the other cross streets. It had a capacity somewhat lower than the other cross streets.

The rates of traffic flow based on the 10-minute periods of maximum volume were frequently greater than the rates based on the loaded cycles. These 10-minute periods included many cycles that were not fully loaded. It is evident, therefore, that greater volumes are sometimes carried during unloaded cycles than during loaded cycles. A cycle was considered to be loaded at an intersection approach if vehicles were traveling through the intersection during the entire green signal period and the stream was interrupted by the red signal. Frequently, when all drivers were alert, accelerated quickly, and allowed a minimum headway, all vehicles waiting at the intersection and those arriving during the green interval were able to clear the intersection before the green time expired. A cycle of this type would not be classed as a loaded cycle. At other times one or two slow drivers or some other impediment tended to reduce the movement of vehicles through the intersection and, as a result, the cycle was classified as being fully loaded despite the fact that the number of vehicles counted was less than during other cycles which were not so classified.

Capacities unusually high

The estimated possible capacity of 13th Street operating one way northbound is 7,000 vehicles per hour of green. This is equivalent to 1,750 vehicles per hour of green per 10 feet of width or per lane. This extremely high volume is only 2 percent below the maximum 10-minute volume observed on 13th Street at Park Road. At the time there was little delay to traffic, although some vehicles required more than one signal cycle to clear the intersection. Periodic traffic counts obtained by automatic recorders indicate that traffic volumes of 3,200 vehicles during 1 hour occur frequently. This is equivalent to 6,400 vehicles per hour of green at Park Road. Thus 13th Street often carries a volume for a 1-hour period which is in excess of the peak-hour volume recorded during this study.

The estimated capacity of 3,600 vehicles per hour of green for each of the one-way streets crossing 13th is also unusually high. This is equivalent to 1,200 vehicles per hour of green per 10 feet of surface width,

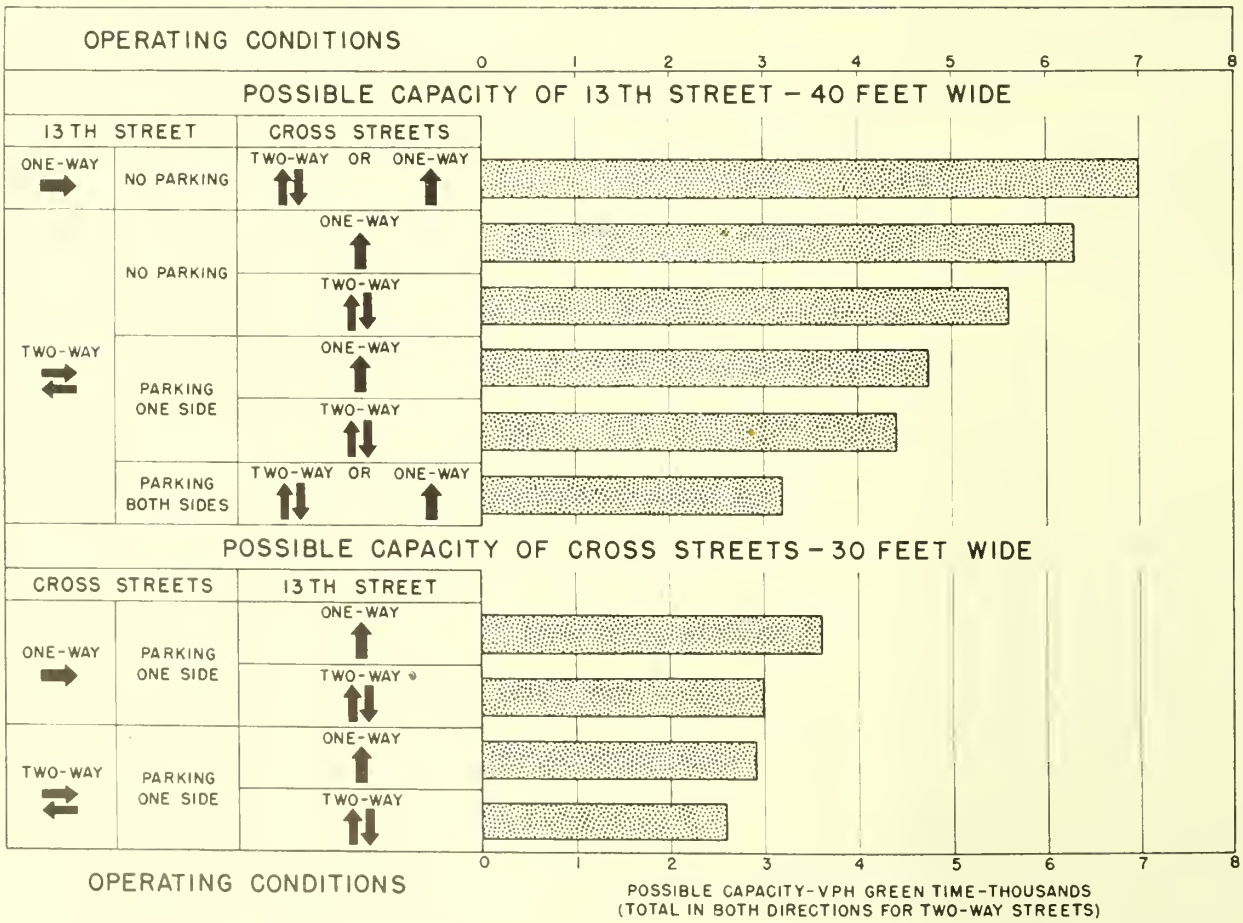

Figure 4.—Street capacity with signal control related to directional operation and parking conditions.

curb to curb, with parking on one side. The capacity of each of the two-way streets crossing 13th Street is also unusually high.

These very high capacities certainly exceed those attained on most city streets. At these high capacities the flow of traffic through an intersection is likely to be greatly affected by a slight change in signal timing, especially in the time offset between succeeding signals which is very critical in a progressive system. Minor accidents, stalled vehicles, and severe weather conditions also result in sharp reductions in capacity. Capacity values considerably lower than those found on 13th Street must therefore be used for design when planning one-way street systems and other improvements to city traffic facilities, or it is likely that adequate capacity will not be provided.

Directional operation and parking related to capacity

By comparing the possible capacities of the approaches, the effect that two-way and one-way operation has on capacity was determined for both 13th Street and the cross streets. The effect of parking on the capacity of 13th Street while operating as a two-way street was also determined by comparing the capacity in the northbound direction with the capacity in the southbound direction. Southbound vehicles could park on the west side of the street while parking was prohibited on the east side. The effects of these conditions are shown graphically in figure 4.

Figure 4 shows that under one set of conditions the possible capacity of 13th Street is only 3,200 vehicles per hour of green signal time, whereas under another set of conditions the possible capacity is 7,000 vehicles per hour of green or 2.19 times as high as the former. The improvement in capacity was realized by eliminating parking and changing from two-way to one-way operation on 13th Street. Likewise, the capacity of a cross street in one case is 2,600 vehicles per hour of green time and in another case, 3,600 vehicles per hour of green. The difference of 1,000 vehicles per hour or 38 percent is a direct result of changing from two-way to one-way operation on both of the intersecting streets and does not involve a change in the parking condition on the cross street.

One of the most important results of the study is illustrated by figure 4, which shows that the operating conditions on one street affect not only the capacity of that street but also the capacity of the intersecting street. Under certain conditions, for example, the capacity of 13th Street is higher when the cross street carries one-way traffic than when it carries two-way traffic. The same is true for the capacity of the cross streets in relation to the directional flow on 13th Street.

Table 4 shows the effect on intersection capacity of each change that was made on 13th Street and the cross streets with respect to directional operation and parking conditions. In this table, 13th Street is referred to as the major street and the cross streets are referred to as minor streets. By using this terminology the results can be more directly compared with the results of studies at other locations and more readily applied to similar situations elsewhere.

Table 4.—Percentage increase in capacities of streets at signalized intersections after changing from two-way to one-way operation and after eliminating parking on the major street [1]

Change in operating conditions	Percentage increase in possible capacity for—	
	Major street	Minor street
CHANGING FROM TWO-WAY TO ONE-WAY OPERATION		
Major street, one way with no parking:		
Minor street, two way	25	12
Minor street, one way	11	20
Minor street, one way:		
Major street, two way with parking on both sides	0	15
Major street, two way with parking on one side	8	15
Major street, two way with no parking	12	15
Major street, one way with no parking	0	24
ELIMINATING PARKING ON MAJOR STREET [2]		
From one side:		
Minor street, one way	48	----
Minor street, two way	37	----
From both sides:		
Minor street, one way	97	----
Minor street, two way	75	----

[1] Parking permitted on one side of minor street in all cases.
[2] Two-way traffic on major street in all cases.

It is of particular interest that a change from two-way to one-way operation of traffic on the major street increased the capacity of the minor street as well as the capacity of the major street. The increase for the major street was 25 percent at locations where the minor street was two way and only 11 percent where the minor street was one way. This same change also increased the capacity of the one-way minor streets by 20 percent and the capacity of the two-way minor streets by 12 percent. Thus, the minor streets that were benefited most were those at locations where the major street benefited least by a change in its directional operation.

The cross streets with one-way traffic had a greater capacity than those with two-way traffic. The difference in capacity was greater (24 percent compared with 15 percent) when the major street was also one way rather than two way. This difference in operation did not, however, benefit the capacity of the major street when it carried two-way traffic with parking on both sides or one-way traffic with no parking. One-way operation on the cross streets was of some benefit to the major street when it carried two-way traffic and parking was eliminated from one or both sides.

Eliminating parking on the major street had a far greater effect on its capacity than changing from two-way to one-way operation. The elimination of parking from both sides of the major street nearly doubled its capacity—an increase of 97 percent at intersections with one-way minor streets. At intersections with two-way minor streets, the increase was 75 percent. Eliminating parking on one side of the major street had about half the effect of eliminating parking on both sides.

It should also be pointed out that progressive movement, which reduces travel time, is usually more easily attained on a one-way than on a two-way street. This is an additional and very important advantage of one-way operation.

Stop-Sign Control on Cross Streets

All cross streets were operating at their possible capacities during the rush period on the day that stop signs were used only on the

Table 5.—Traffic volumes and delay per vehicle with stop-sign control at approaches to 13th Street and no control on 13th Street

Location	Period of study [1]	Average for study			Average for peak 10-minute period		
		Traffic volume		Delay per vehicle on cross street	Traffic volume		Delay per vehicle on cross street
		Cross street	13th Street		Cross street	13th Street	
CROSS STREETS CARRYING TWO-WAY TRAFFIC							
	Minutes	*V.p.h.*	*V.p.h.*	*Minutes*	*V.p.h.*	*V.p.h.*	*Minutes*
13th Street, two way:							
Park Rd., eastbound	32	272	1,140	0.34	336	1,128	0.43
Park Rd., westbound	32	248	1,140	.48	276	1,140	.61
Irving St., eastbound	38	207	1,265	.41	234	1,236	.15
Irving St., westbound	38	111	1,265	.46	114	1,236	.74
13th Street, one way:							
Park Rd., eastbound	60	322	1,658	2.89	366	1,950	3.21
Park Rd., westbound	60	228	1,658	.51	276	1,746	.74
Irving St., eastbound	60	245	2,172	1.67	318	3,126	1.25
Irving St., westbound	60	143	1,927	.80	168	2,490	.71
CROSS STREETS CARRYING ONE-WAY TRAFFIC							
13th Street, two way:							
Columbia Rd	40	491	1,270	0.59	570	1,260	0.63
Harvard St	34	434	1,200	.63	462	1,152	.70
13th Street, one way:							
Columbia Rd.	60	572	1,915	3.81	738	1,866	4.78
Harvard St	60	744	1,785	.92	882	2,070	.64

[1] Where studies exceeded 1 hour, data for the 1-hour period of maximum traffic flow were used. Studies of less than an hour were for the periods immediately preceding the change to one-way operation on 13th Street.

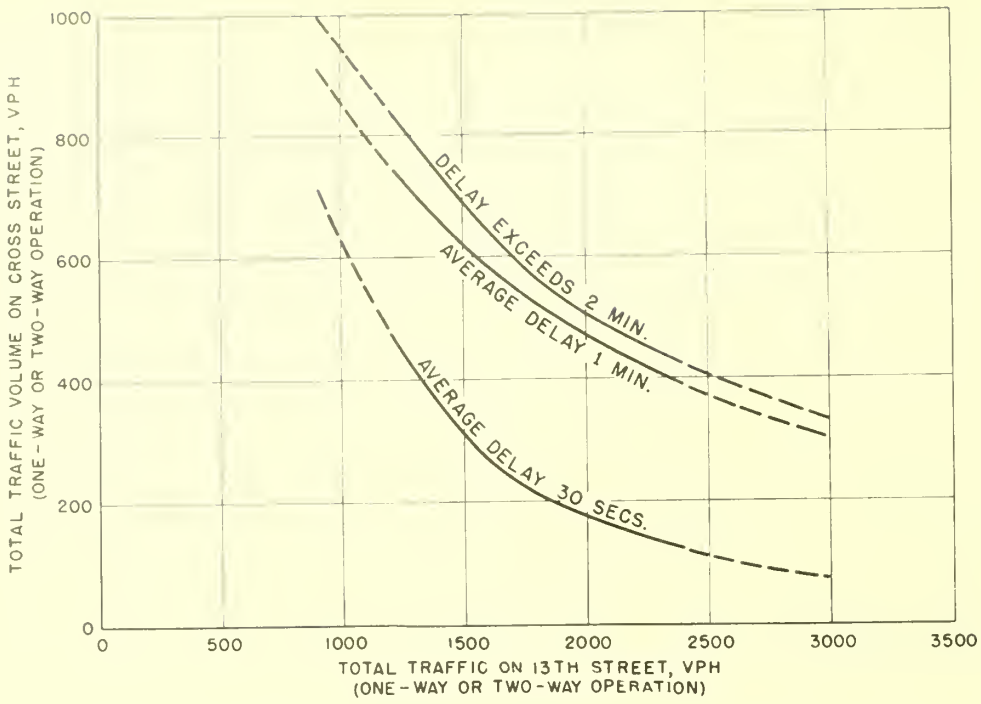

Figure 5.—Delay to traffic on cross streets controlled by stop signs and with no control on 13th Street.

cross streets. Long queues of waiting vehicles developed, and occasionally it was necessary for a police officer to clear the backup on the cross streets. Vehicles on the cross streets were aided somewhat by the signals at intersections on the through street north and south of the study area which caused gaps in traffic on 13th Street.

Table 5 shows the observed traffic volumes and the delays to traffic on the cross streets at the intersections studied. Volumes and delays are listed for the 1-hour and for the 10-minute periods of heaviest cross-street traffic while 13th Street was one way and also while it was two way.

Similar information was obtained for all 10-minute periods during the studies, and the average delays to cross-street traffic were compared for various traffic volumes on the cross streets and on 13th Street. As was expected, the delay to cross-street traffic was found to increase with an increase in the total traffic volume on 13th Street as well as with an increase in the total cross-street traffic. A less expected finding was that the delay to cross-street traffic was independent of the directional usage of either 13th Street or the cross street. Consequently, the delay to cross-street traffic when controlled by stop signs can be shown on one graph representing both two-way and one-way operation on both of the intersecting streets. This has been done in figure 5.

The two lower curves of figure 5 show the combinations of sustained through-street and stop-street traffic volumes which produced average delays to cross-street traffic of 30 seconds and of 1 minute per vehicle. The total traffic volume on 13th Street is shown on the horizontal axis, and the total traffic volume on the cross street is shown on the vertical axis. For example, a cross flow of about 600 vehicles per hour can be accommodated with an average delay of 1 minute when the total traffic volume on 13th Street is 1,600 vehicles per hour. Combinations of volumes which lasted for only short periods sometimes caused delays much longer or shorter than those indicated by the curves. If the volumes were sustained for 20 or 30 minutes, however, the delays were as indicated by the curves.

The third curve in figure 5 shows the maximum volumes that the cross streets can accommodate with various volumes on 13th Street. The delay accompanying these volumes was at least 2 minutes and for any given combination of volumes might have been several minutes. Once the traffic volumes on the cross street and on 13th Street became sufficiently great to cause a delay of 2 minutes, the delay could increase to several minutes within a short period of time with no change in the traffic volumes. This curve therefore represents the traffic volumes for all delays above 2 minutes. It thus represents the possible capacities of the cross streets when controlled by stop signs and with no control on 13th Street.

Volumes which cause a 30-second delay can be exceeded by as much as 100 percent without increasing the delay to more than 1 minute. Volumes which result in a 1-minute average delay, however, can only be exceeded by about 10 percent without increasing the delay to more than 2 minutes.

The small difference between the volumes shown for the curve representing a 1-minute delay and the curve representing a delay in excess of 2 minutes indicates that at these intersections the critical volumes are those producing a delay of between 1 and 2 minutes per vehicle to cross-street traffic.

The volume combinations indicated by the 30-second delay curve fit the usual definition of practical capacity for the prevailing roadway and traffic conditions since greater delay and restriction to movement would appear unreasonable to most drivers. With the normal setting of the signals, average delays of 30 seconds or more to minor-street traffic were infrequent even at the highest volumes observed. In another study it was found that drivers are unwilling to accept longer delays at stop signs than at signals.[4] It is concluded that the practical capacity of the cross street when controlled by stop signs with no control on the through street is represented by the curve for an average delay of 30 seconds.

Stop signs compared with signals

Figure 6 shows the possible and practical capacities of the cross streets with stop-sign control on these streets and the possible capacities with signal control. The curves for stop-sign control represent both one-way and two-way operation and are based on the delay to cross-street traffic. With stop signs on the cross streets, 13th Street could undoubtedly carry as much traffic as with signals but not without an unreasonable delay to cross-street traffic.

For the signals, separate curves are shown for one-way and two-way operation, but separate curves are not shown for possible and practical capacities. With the progressive signal control there was no appreciable increase in the delay to traffic as the traffic increased from comparatively low volumes to those at possible capacity. Thus it was not feasible to establish a value for practical capacity with progressive signal control on the basis of delay to traffic. For the purpose of comparing the three different types of control on the basis of a tolerable delay to traffic, the curves for practical capacity with stop-sign control should be compared directly with the possible capacity curves for signal control.

It may be noted that the curves for signal control intersect the X axis and the Y axis at values below those given in the last column of tables 2 and 3. This is because the amber time in each cycle was not included in the green time when calculating the traffic volumes for the signal-control curves.

From figure 6 it may be seen that both the possible and the practical capacities of the cross streets are much lower with stop-sign control than with signal control. This is especially true with one-way operation on both streets. Even with volumes as low as 1,000 vehicles per hour on 13th Street and with two-way traffic on both streets, the possible capacity of the cross streets with stop signs was about one-half the capacity with signal control. With one-way traffic and at other volumes on 13th Street, the difference was even greater.

[4] *Effects of reversible lane movement signalization of three-lane highways*, by M. Mansfield Todd. Proceedings of the Highway Research Board, vol. 30, 1950, pp. 346-354.

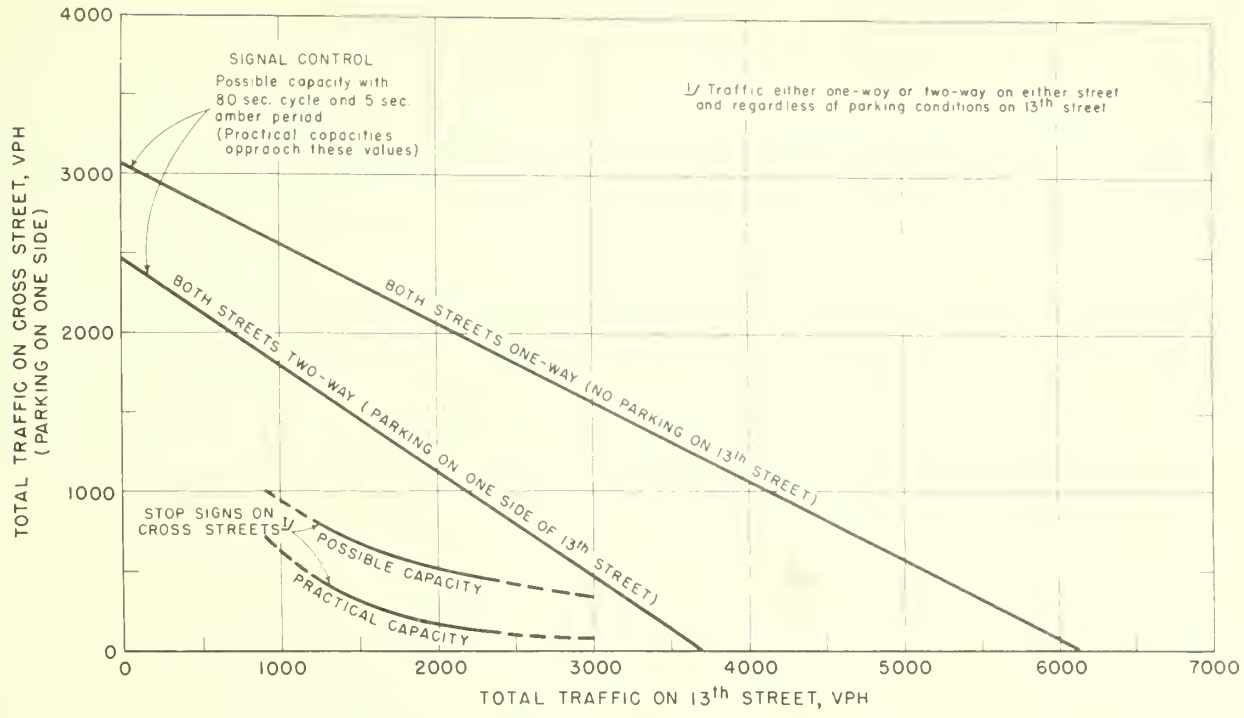
SIGNAL CONTROL
Possible capacity with 80 sec. cycle and 5 sec. amber period (Practical capacities approach these values)
1/ Traffic either one-way or two-way on either street and regardless of parking conditions on 13th street
BOTH STREETS ONE-WAY (NO PARKING ON 13th STREET)
BOTH STREETS TWO-WAY (PARKING ON ONE SIDE OF 13th STREET)
STOP SIGNS ON CROSS STREETS 1/
POSSIBLE CAPACITY
PRACTICAL CAPACITY
TOTAL TRAFFIC ON CROSS STREET, VPH (PARKING ON ONE SIDE)
TOTAL TRAFFIC ON 13th STREET, VPH
0
1000
2000
3000
4000
5000
6000
7000

Figure 6.—Capacities for two types of intersection control.

At all traffic volumes there was practically no delay on the through street when stop signs were used to control traffic on the cross streets. Cross-street traffic experienced some delay even at the lower volumes, however, and this delay increased rapidly as the volume increased. With signal control both 13th Street traffic and cross-street traffic experienced some delay, yet this delay at all observed volumes was never greater for the cross streets than the delay with stop signs. The delay with signals was small as a result of the well-coordinated progressive system operating on the cross streets as well as on 13th Street.

While the difference in delay with stop signs and with signals was small at low volumes, it became very large at the higher volumes. For example, when the traffic volume on the through street approached 3,000 vehicles per hour, the delay to traffic on the cross street varied from 1½ to 6 minutes per vehicle with stop signs on the cross street, compared with 20 to 40 seconds per vehicle with signals. At a volume of 1,800 vehicles per hour on the through street and a cross-street volume of about 250 vehicles per hour, the average delay to all traffic on both streets was 10 seconds per vehicle with cross-street traffic controlled by stop signs and also when traffic was controlled by signals. At lower volumes the delays were only slightly less and were about equal for both types of control. At volumes above 1,800 vehicles per hour on the through street, the average delay to all traffic was greater with stop-sign control on the cross streets than with signal control.

Traffic on the through street was protected by stop signs on the cross streets, and had it not been impeded at other intersections along the route, the volume of traffic could theoretically have increased to 7,000 vehicles per hour. This type of control is not feasible, however, for such heavy traffic volumes on the through street because cross traffic is almost completely blocked.

When an intersection at which traffic on one of the streets is controlled by stop signs is adjacent to an intersection where traffic on the through street is controlled by signals, the capacity of the cross street is somewhat greater than it would be if there were no traffic signals in the immediate vicinity. The signal at the neighboring intersection creates gaps in the stream of traffic on the through street into which vehicles waiting at the stop sign may enter. At such locations traffic volumes greater than those indicated by the curves in figure 6 could probably be discharged from cross streets controlled by stop signs.

The traffic volumes observed in this study are evidently too great to be handled satisfactorily at intersections controlled by stop signs on the cross street. Even with moderately heavy volumes on the cross streets, this traffic was delayed as much or more by stop signs than by signals. At the higher volumes the delay with stop signs definitely became intolerable to cross-street traffic, although through traffic was completely unrestricted.

Stop-Sign Control on All Approaches

When stop signs were used on all approaches to the three intersections studied with this type of control, they were installed on the near side as well as the far side of the intersections, and advance warning signs were erected. On the one-way streets, signs were mounted on the left as well as the right-hand side of the street. Even under these conditions, it was necessary to discontinue the studies when the traffic volume reached the practical capacity of the intersections. The study had to be terminated after 13th Street had been operating one way for little more than an hour and before the height of the afternoon rush was reached. The frequency with which a police officer was required to regulate the various movements immediately before the study was terminated made it apparent that the practical capacities of the intersections for this type of control were reached.

Table 6 shows traffic volumes for the heaviest hour and for the heaviest 10-minute period of the study with stop signs on all intersection approaches. For each intersection, the volumes are shown separately for 13th Street and for the intersecting or cross street, and the combined total for the two is shown in the column, "Total for intersection." The average delay per vehicle during the period when the particular volume occurred is also shown in table 6. The peak volume on 13th Street did not always occur during the same time period as that for the cross street, and for this reason each intersection is listed twice in the table. The volumes for the cross streets as shown in the upper portion of the table are those observed during the period when 13th Street was carrying its peak load. In the lower portion of the table the volumes shown for 13th Street are those observed when the cross streets were at a peak.

The highest 10-minute volume was observed at the intersection of 13th Street and Columbia Road, with 13th Street operating as a one-way street northbound and Columbia Road, one-way westbound. The rate of flow on 13th Street during this period was 2,280 vehicles per hour, while the rate on Columbia Road during the same period was 762 vehicles per hour. The total volume for the intersection,

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Table 6.—Traffic volumes and delay per vehicle with stop-sign control on all approaches

Location	1-hour period						10-minute period					
	13th Street		Cross streets		Total for intersection		13th Street		Cross streets		Total for intersection	
	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay	Traffic volume	Average delay
During Peak Volume On 13th Street [1]												
	V. p. h.	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*	*V. p. h.*	*Minutes*
13th Street, two way:												
At Park Road	1,231	0.44	512	0.54	1,743	0.47	1,476	0.31	642	0.39	2,118	0.33
At Columbia Road	1,400	.41	567	.27	1,967	.37	1,602	.49	480	.28	2,082	.44
At Harvard Street	1,389	.49	492	.47	1,881	.48	1,542	.47	594	.42	2,136	.46
13th Street, one way:												
At Park Road	1,502	.23	691	.80	2,193	.41	1,794	.25	810	1.05	2,604	.50
At Columbia Road	1,615	.31	668	.42	2,283	.34	2,280	.43	762	.52	3,042	.45
At Harvard Street	1,481	.38	653	.60	2,134	.45	1,890	.26	696	.86	2,586	.42
During Peak Volume on Two-Way Cross Street												
13th Street, two way:												
Park Road	1,147	0.35	565	0.54	1,712	0.41	1,476	0.31	642	0.39	2,118	0.33
13th Street, one way:												
Park Road	1,502	.23	691	.80	2,193	.41	1,794	.25	810	1.05	2,604	.50
During Peak Volumes on One-Way Cross Streets												
13th Street, two way:												
Columbia Road	1,400	0.41	567	0.27	1,967	0.37	1,314	0.36	684	0.37	1,998	0.36
Harvard Street	1,387	.53	495	.49	1,882	.52	1,530	.41	600	.36	2,130	.40
13th Street, one way:												
Columbia Road	1,615	.31	668	.42	2,283	.34	1,968	.45	822	.70	2,790	.52
Harvard Street	1,481	.38	653	.60	2,134	.45	1,818	.25	858	.45	2,676	.31

[1] Parking permitted on one side of 13th Street during two-way operation; parking prohibited during one-way operation.

3,042 vehicles per hour, is equivalent to an average of 507 vehicles per lane per hour for all lanes entering the intersection. The accompanying delay averaged 31 seconds per vehicle to traffic on the cross street and 26 seconds per vehicle to traffic on the through street. This is very close to the delay of 30 seconds which was used as the criterion for practical capacity with stop-sign control on the cross streets. It seems reasonable to assume that most drivers would consider a delay greater than this intolerable at an all-way stop intersection just as they do at a cross street with stop signs.

During the 10-minute periods of maximum traffic flow reported in table 6, it is noteworthy that the average volume per lane, including all approaches, was usually between 400 and 500 vehicles per hour. Most of the average delays to cross-street traffic observed during these same periods vary between 21 and 52 seconds. This indicates that the traffic load was between the practical and possible capacities of the intersections. It seems reasonable to assume that somewhat higher volumes might be carried with no increases in delay after a period of familiarization for the drivers. The practical capacity of these intersections with stop-sign control on all approaches and under the other existing conditions is therefore

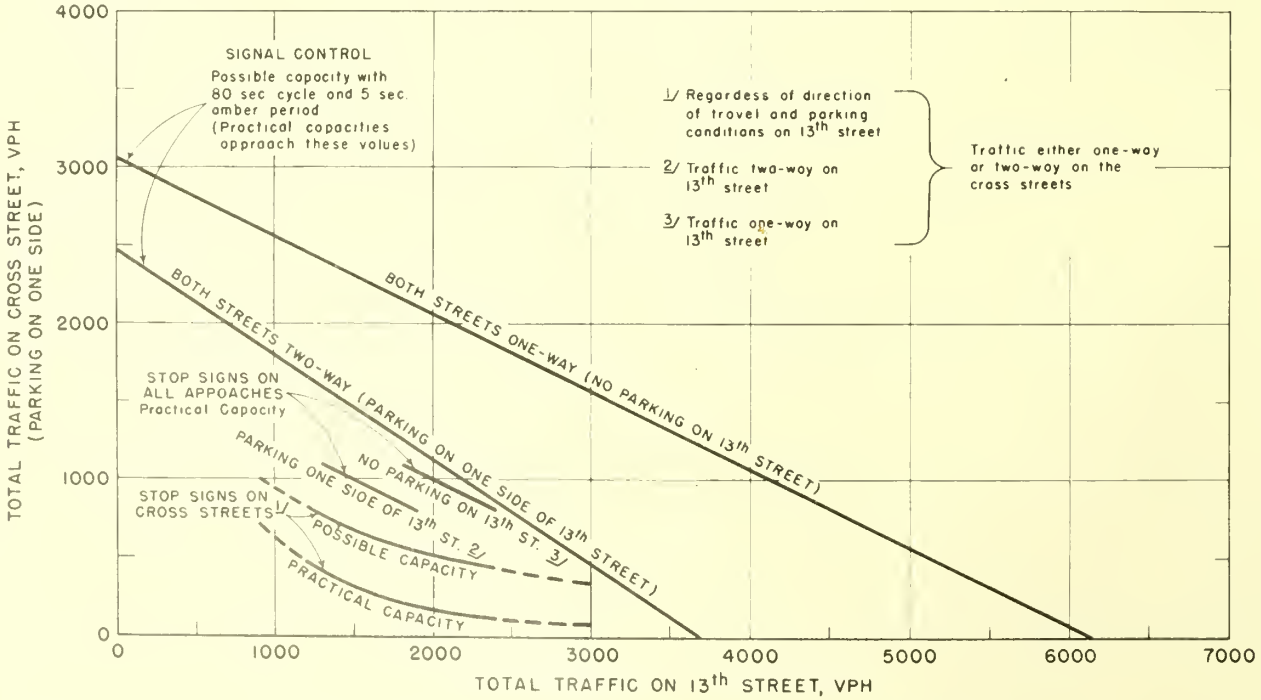
SIGNAL CONTROL
Possible capacity with 80 sec cycle and 5 sec. amber period (Practical capacities approach these values)
1/ Regardless of direction of travel and parking conditions on 13th street
2/ Traffic two-way on 13th street
3/ Traffic one-way on 13th street
Traffic either one-way or two-way on the cross streets
BOTH STREETS ONE-WAY (NO PARKING ON 13th STREET)
BOTH STREETS TWO-WAY (PARKING ON ONE SIDE OF 13th STREET)
STOP SIGNS ON ALL APPOACHES Practical Capacity
PARKING ONE SIDE OF 13th ST. 2/
NO PARKING ON 13th ST. 3/
STOP SIGNS ON CROSS STREETS 1/
POSSIBLE CAPACITY
PRACTICAL CAPACITY
TOTAL TRAFFIC ON CROSS STREET, VPH (PARKING ON ONE SIDE)
TOTAL TRAFFIC ON 13th STREET, VPH
0
1000
2000
3000
4000
5000
6000
7000

Figure 7.—Capacities for three types of intersection control.

somewhere near 500 vehicles per hour for each lane used by traffic entering the intersection. This capacity is based on a reasonable traffic delay but does not take into consideration pedestrian delays or accident hazards which would probably tend to reduce the 500 figure to a somewhat lower value. The few pedestrians at the intersections studied experienced long delays in crossing the streets even at moderate volumes.

Practical capacity for all-way stop control is compared with the capacities determined for the other types of control in figure 7. This shows that under the traffic conditions prevailing at these intersections the capacity with all-way stop control is greater than with cross-street stop-sign control and less than with signal control. As previously stated, the capacities with stop signs on the cross streets are limited to those that permit a reasonable movement of the cross-street traffic.

At intersections where both streets are two way, the practical capacity with four-way stop signs approaches the capacity with signals. The capacity with signals is much greater, however, than with stop signs on all approaches when both streets are one way.

The delay to traffic was compared for the three types of control. At the traffic volumes observed, the delay to all traffic was found to be more with stop signs on all approaches than with signal control. Also, when the volume on the through street was below 1,800 vehicles per hour, stop signs on all approaches resulted in a greater average delay to all traffic than did stop signs on the cross streets. When the volume on the through street was greater than 1,800 vehicles per hour, the average delay to all traffic was less with stop signs on all approaches than with stop signs on the cross streets.

An analysis of the delays shown in tables 5 and 6 indicates that the delay to cross-street traffic is far less when stop signs are used on all approaches than when used on the cross street only. An examination of other data obtained for lower traffic volumes shows this also to be the case for the lower volumes. The cross-street volumes in one direction during this study were never below 100 vehicles per hour. For cross-street volumes lower than this figure the relation between delay and type of stop-sign control might be quite different from that found in this study.

Use of the Kelly Ball as a Device for Measuring the Consistency of Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by WILLIAM E. GRIEB and ROBERT A. MARR, JR.
Highway Physical Research Engineers

The Kelly ball test is a simple field method for determining the consistency of plastic concrete. It is made by measuring the penetration of a 30-pound metal "ball" into the surface of the concrete. This test can be made on the concrete in place which results in a considerable savings in time and effort compared with the slump test. The data reported in this article show good correlation between the slump and Kelly ball penetration tests.

THE American Society for Testing Materials standard slump test [1] has been used for many years as a measure of the consistency of fresh concrete in the laboratory and on the job. As a laboratory procedure it is reasonably satisfactory. In the field, especially on paving work, it has several disadvantages and the most serious of these is the time required to make the test. Others are the necessity for careful selection of samples and the close attention to details of the technique required to obtain reasonably accurate results.

Realizing the disadvantages of the slump test as a field control method, Prof. J. W. Kelly of the Department of Civil Engineering, University of California, developed a penetration device that is rugged and portable. This test was recently made ASTM Tentative Standard C 360–55T.

Professors Kelly and Polivka of the University of California gave the following account of the development of this test in an article published recently.[2]

> The test was developed in the Engineering Materials Laboratory of the University of California at Berkeley as an outgrowth of an attempt to devise a simple test for workability of

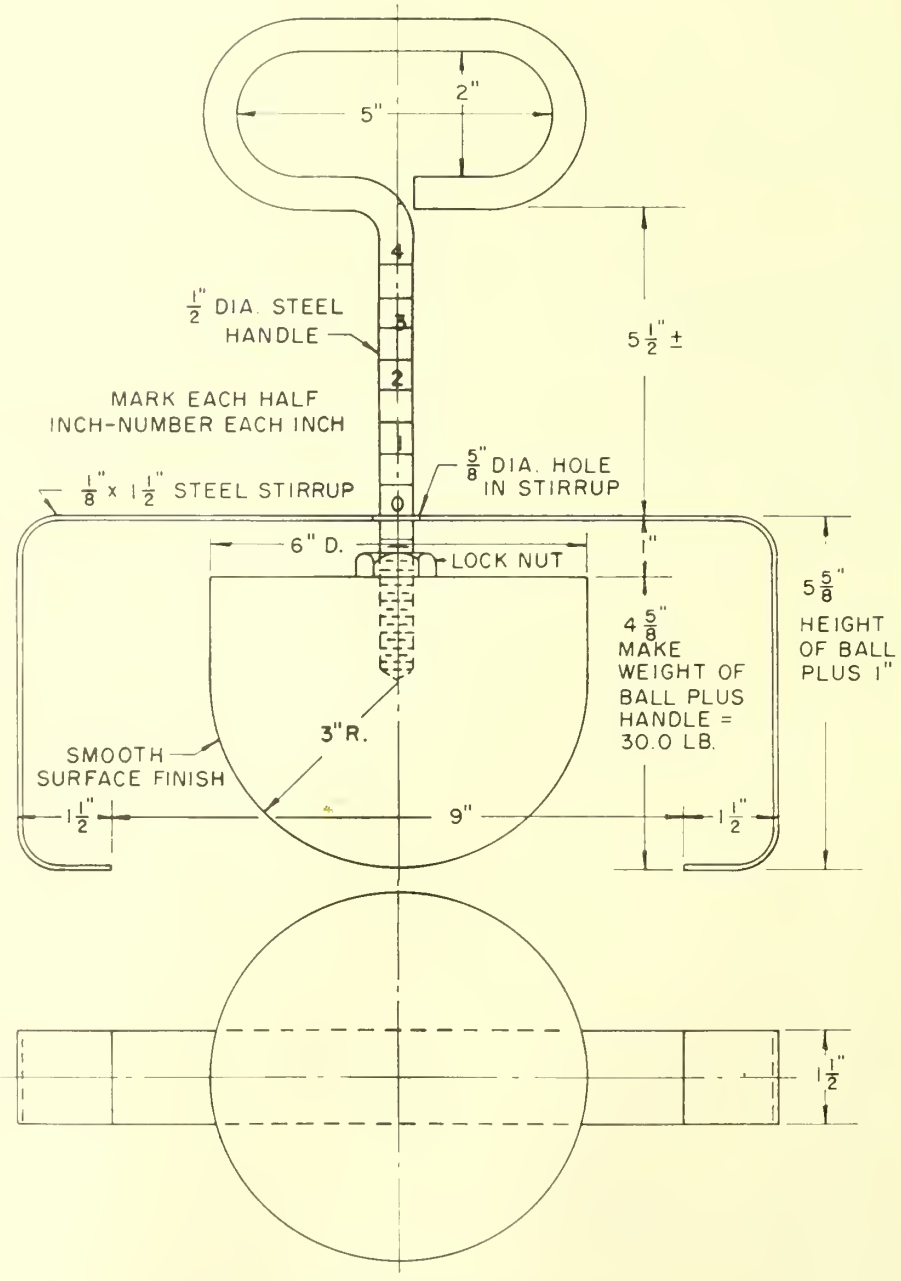

Figure 1.—Sketch of the Kelly ball apparatus used for measuring the consistency of concrete.

[1] *Standard method of test for slump of portland-cement concrete*, ASTM Designation: C 143–52. ASTM Standards, 1952, Part 3, pp. 1086–1087.

[2] *Ball test for field control of concrete consistency*, by J. W. Kelly and Milos Polivka. Journal of the American Concrete Institute, May 1955, vol. 26, No. 9, pp. 881–888.

concrete. Workability is an elusive property, and early trials with various balls showed little correlation with the more elaborate tests in laboratory use. However, it was observed that static ball penetration correlated rather well with slump and it became apparent that the penetration test measured some property similar to slump—a property which had been termed "consistency" but which is now called merely "slump" in ASTM C 143. It is the significant property which is measurable in the field for practical purposes.

A 6-inch ball was considered to be the smallest that would integrate the resistance to penetration over several pieces of aggregate, and a 30-pound weight was found to be the lightest that would penetrate reproducibly the stiffest mixes of plastic concrete. This combination of area (or displaced volume) and force has been found applicable even to harsh concrete containing 2½-inch aggregate and having a nominal slump, when wet-screened, of 1½ inches. The apparatus has also been used on mass concrete containing 0 to 6-inch crushed aggregate by making the penetration test only on areas which had been found by probing to be free from the larger pieces of aggregate. A 20-pound weight on a 6-inch ball has sometimes been used for lightweight concrete.

Penetration tests have been developed independently in other countries. A static test used in Spain employs a weight on a spherical tip and having a flared edge so that the plunger will not sink too deep into wet concrete. The German Committee on Reinforced Concrete has adopted an impact test suitable for stiff mixes or mixes of low cement content; it consists in dropping a 33-pound plunger having a 4-inch hemispherical tip 8 inches onto the surface. In England, the Wigmore consistometer employs a metal ball set on the surface of a concrete sample which is vibrated on a table.

* * * * *

The static ball test was introduced to field use by E. L. Howard, testing engineer, Pacific Coast Aggregates, Inc., San Francisco. His experience with the variety of mixes used in ready-mixed concrete was so successful that it encouraged the authors to report the test to ASTM Committee C-9 at its San Francisco meeting in 1949. Mr. Howard has continued to contribute to its field development, and is convinced that it will eventually replace the slump test.

Many other organizations have adopted the ball test, and hundreds of balls are in use throughout the country. The California Division of Highways has adopted it as a standard for field use on pavement construction. At least two other State highway departments—North Carolina and Colorado—are using it extensively. The Waterways Experiment Station, Concrete Division, U. S. Army Corps of Engineers, has adopted it as an alternative standard.

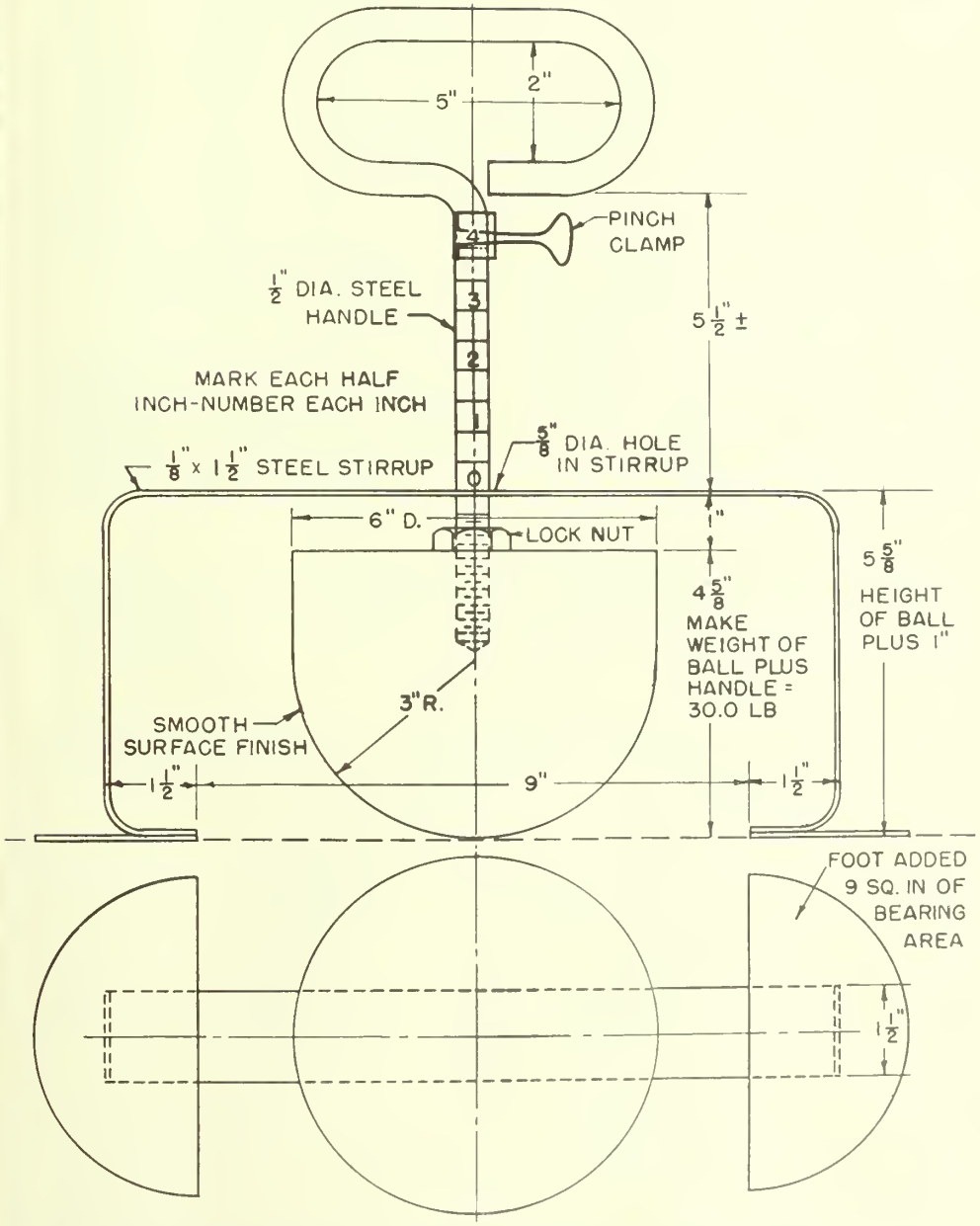

Figure 2.—Sketch of the Kelly ball apparatus after modification by the Bureau of Public Roads.

Kelly Ball Apparatus

The apparatus is popularly known as the "Kelly ball." It is made by machining into a hemisphere, one end of a solid right cylinder 6 inches in diameter and 4⅝ inches in height. It is fitted with a graduated vertical rod ½ inch in diameter which serves as a measuring scale and a handle. The vertical portion of the rod is graduated in ½-inch units with each inch numbered. The ball is guided by a stirrup or frame which also serves as a reference line in measuring penetration of the ball into the plastic concrete. The zero on the graduated handle coincides with the top of the frame when the apparatus rests on a level rigid surface. The weight of the ball and handle is 30 pounds. A sketch of the original apparatus is shown in figure 1.

Modification of the Kelly Ball

The Kelly ball equipment used by the Bureau of Public Roads differs from the original in the following details:

To prevent the reference frame from tilting, the bearing area of each foot of the frame (originally specified as 1½ square inches) was increased by the addition of semicircular bearing plates of 5-inch diameter (area approximately 9¾ square inches). The clear distance between the feet was maintained at 9 inches as originally specified (see fig. 2). This

Figure 3.—Kelly ball penetration test being performed under field conditions.

change is included in the ASTM Tentative Method C 360-55T.

To facilitate reading the depth of penetration of the ball, a vertically movable pinch clamp was fastened to the graduated handle. This is clamped at the top of the handle before making the test and is lowered to make contact with the top of the frame after the ball has penetrated the surface of the concrete. The apparatus is then removed from the concrete and the penetration reading is made by reading the position of the clamp on the rod. This makes the test procedure more convenient for the operator and avoids any possible delay in the concreting operation.

Use of the Kelly Ball

Plastic concrete can be tested with the Kelly ball after placement in the forms and prior to any manipulation, or in suitable containers such as tubs, pans, wheelbarrows, or buggies.

In making the test, the surface of the concrete is smoothed and leveled quickly by the use of a small wooden float or screed. The surface is worked as little as possible to avoid formation of a mortar layer. During the test, the adjacent concrete should not be vibrated, jarred, or agitated.

The ball is held vertically by the handle in very light contact with the leveled concrete surface and with the zero on the rod coincident with the top of the frame. The handle is then released and the depth of penetration of the ball into the concrete is estimated on the graduated rod to the nearest 0.1 inch (see fig. 3). A minimum of three readings should be taken from a batch or location. No correction is made for any slight settlement of the feet of the frame. The test requires less than ½ minute to perform, which allows the operator sufficient time to work where the concrete is being discharged from the mixer without delaying the placing and finishing operations.

Experience has indicated that the minimum depth of concrete tested should be 6 inches for a maximum size of coarse aggregate of 2 inches or less. When larger coarse aggregate is used, the minimum depth should be three times the nominal maximum size of coarse aggregate. When testing concrete placed inside forms as in piers, walls, etc., a minimum horizontal distance of 9 inches from the face of the form to the point tested should be maintained to avoid boundary effects. For concrete discharged on the subgrade in pavement work, no test should be made nearer than 9 inches to the form edge of the leveled surface of the concrete. For a second test in the same batch of concrete, the foot of the stirrup should be at least 6 inches from the point where the foot rested in the first test.

Field Tests

A limited number of tests were made on several paving projects to correlate Kelly ball penetration readings with the corresponding slumps. The concrete used on the first of these projects contained 6½ sacks of cement per cubic yard, 2-inch maximum size gravel, and about 5 percent air. Batches were selected so that tests could be made without delaying the progress of the work, and so that some adjustments could be made in the water content and consistency of the batches. The concrete was discharged on the subgrade in a pile 8 to 10 inches in height. A sample for the slump test was taken and thoroughly remixed in a pan.

The Kelly ball penetration test was made on the concrete in place on the subgrade prior to any manipulation. The top surface of the concrete was leveled with a wood float, the apparatus set on that surface, and the penetration read. The water content of the concrete was varied in order to obtain a greater range in consistency. Usually two or three Kelly ball penetration readings were made for comparison with the reading of each slump. Test data taken over a 2-day period are tabulated in table 1 and are shown graphically in figure 4 (A). The average penetration was 2.4 inches, the average slump was 3.6 inches, and the ratio of penetration to slump was 1 to 1.5.

The concrete on the second project had the same cement content and the same maximum size coarse aggregate as that on the first project. The aggregates were from a different source and the concrete was mixed in a ready-mix truck. The test data taken over a 7-day period are tabulated in table 1 and are shown graphically in figure 4 (B). For this job the average penetration was 2.3 inches and the average slump was 3.2 inches. The ratio of penetration to slump was 1 to 1.4.

The concrete in the third project was similar to that in the second project, but the maximum size of coarse aggregate was 1 inch instead of 2 inches. The test data taken over a 6-day period are tabulated in table 1 and illustrated in figure 4 (C). The average penetration and slump for this job were 2.3 and 3.0 inches, respectively. The penetration slump ratio was 1 to 1.3.

The data in table 1 and figure 4 (field tests) show a reasonable correlation between Kelly ball penetration and slump readings for a range in slump of 1 to 5½ inches. The average Kelly ball reading multiplied by 1.4 would provide a fair estimate of the corresponding slump for the range indicated and materials used. These limited data indicate that for a maximum size of coarse aggregate of 1 inch this ratio might be reduced to 1.3.

Laboratory Tests

Slump tests and Kelly ball penetration readings were also made on concrete mixed in the laboratory and placed in steel forms for the fabrication of slabs for structural tests containing approximately 33 cubic

Table 1.—Correlation of Kelly ball penetration test and slump test for consistency of concrete under field conditions [1]

Project No. 1			Project No. 2			Project No. 3		
Kelly ball penetration		Slump test	Kelly ball penetration		Slump test	Kelly ball penetration		Slump test
Individual tests	Average, all tests		Individual tests	Average, all tests		Individual tests	Average, all tests	
Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
1.1, 1.2, 1.2	1.2	2.3	1.6	1.6	2.7	1.6	1.6	1.8
1.4, 1.4, 1.1	1.3	.7	1.6, 1.6	1.6	2.0	1.6, 1.5	1.6	1.8
1.8, 1.0	1.4	1.7	1.6, 1.7	1.6	2.6	1.8, 1.6	1.7	2.7
1.4, 1.6	1.5	2.6	1.6, 2.1	1.8	2.4	1.8, 1.9, 1.8	1.8	2.2
1.7, 1.9, 1.5	1.7	2.3	2.0, 1.6	1.8	2.2	2.0, 1.7	1.8	2.7
2.0	2.0	3.3	2.3, 1.3	1.8	2.5	1.9, 2.4, 1.8	2.0	2.7
2.2, 2.2, 2.4	2.3	3.7	2.0, 1.8	1.9	2.8	2.0, 1.9, 2.2	2.0	3.4
2.5, 2.6, 1.9	2.3	3.9	1.7, 2.2	2.0	3.1	2.1, 2.2, 2.0	2.1	2.8
2.3, 2.5	2.4	3.7	2.1	2.1	4.0	2.1, 2.4	2.2	2.8
2.5, 2.4	2.4	4.7	2.2, 2.0	2.1	3.0	2.3, 2.2	2.2	2.5
2.6, 2.8	2.7	3.7	2.2, 2.2	2.2	2.8	2.4, 2.2	2.3	2.6
2.7	2.7	4.3	2.2, 2.2	2.2	3.0	2.3, 2.5	2.4	2.8
2.8	2.8	4.0	2.1, 2.8	2.4	2.8	2.8, 2.8	2.8	3.3
2.7, 3.5, 2.6	2.9	4.7	2.6, 2.1	2.4	3.6	3.0, 2.6	2.8	3.8
3.2, 3.4	3.3	4.5	2.4, 2.6	2.5	3.4	2.8, 2.9, 3.1	2.9	4.3
3.7, 3.1	3.4	4.1	2.8, 2.5	2.6	3.4	3.4, 3.0	3.2	3.8
3.7	3.7	4.4	3.1, 2.0	2.6	3.0	3.8, 4.0	3.9	4.6
4.2, 4.7, 3.7	4.2	5.3	2.5, 2.9	2.7	3.6	----------	------	----
---- ----------	------	----	2.8, 2.9	2.8	5.5	----------	------	----
---------- ------	------	----	3.5, 3.3	3.4	4.8	----------	------	----
---------- ------	------	----	3.6, 3.4	3.5	4.5	----------	------	----
Average [2]	2.4	3.6	----------	2.3	3.2	----------	2.3	3.0

[1] Mixture used for project Nos. 1–2: 2-inch maximum size gravel, 6½ sacks of cement per cubic yard, and 5 percent air content. Same cement content and air content used for project No. 3; maximum size gravel was 1 inch.

[2] Ratios of penetration to slump: project No. 1, 1:1.5; project No. 2, 1:1.4; and project No. 3, 1:1.3.

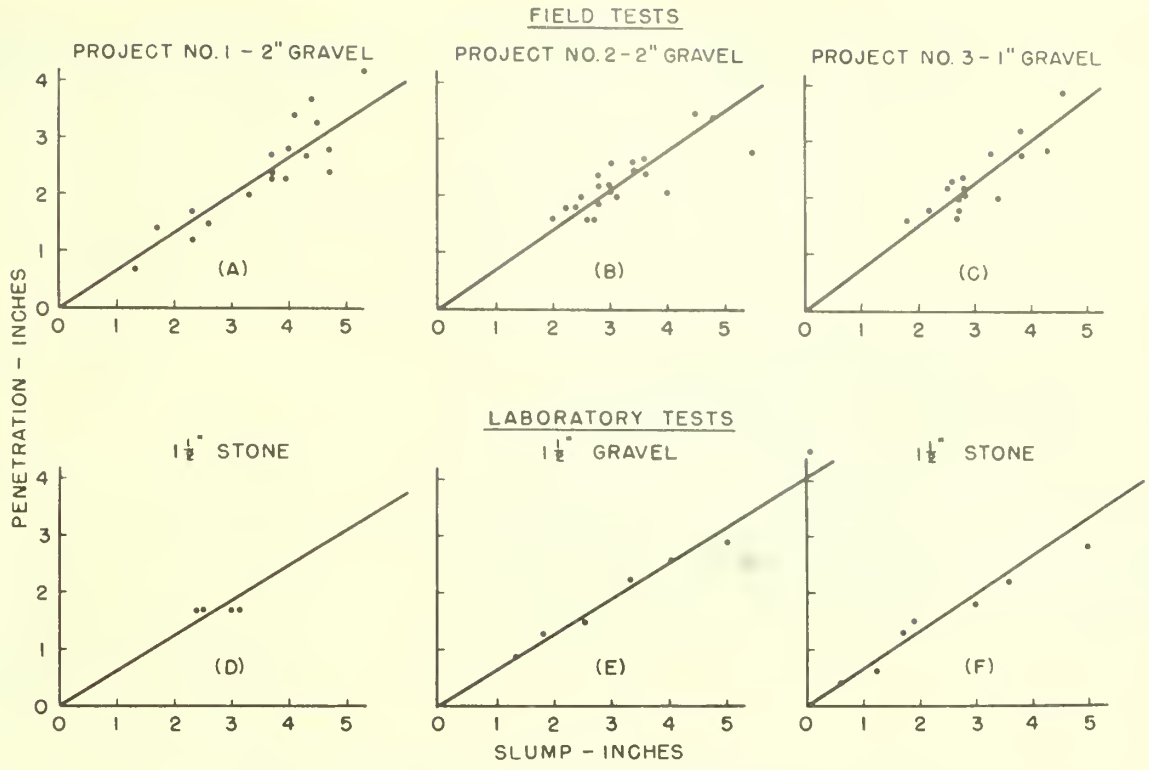

Figure 4.—Correlation of Kelly ball penetration and slump tests.

feet of concrete. The water content was the same for all batches. The concrete was non-air-entrained and contained 6 sacks of cement per cubic yard with crushed-stone coarse aggregate of 1½-inch maximum size. The results of these tests are shown in table 2 and in figure 4 (D). This figure does not have much significance due to the limited number of tests. For these tests the average slump was 1.6 times the Kelly ball reading.

Laboratory tests were made on concrete mixes using both gravel and crushed-stone coarse aggregates of 1½-inch maximum size and having variable slumps. The concrete was non-air-entrained and contained 6 sacks of cement per cubic yard. The water content was varied to produce slumps ranging from 1 to 6 inches. The results of these tests are tabulated in table 2 and are shown graphically in figure 4 (E) for gravel and figure 4 (F) for crushed stone. The ratios of the average Kelly ball penetration to slump were 1 to 1.6 for crushed stone and 1 to 1.5 for gravel. These tests show approximately the same relation between the slump and Kelly ball readings as were shown for the field tests where similar materials were used.

The results of field and laboratory tests discussed in this article are in reasonably good agreement with those obtained by other investigators. In an extensive series of tests conducted by the Concrete Division, Waterways Experiment Station of the U. S. Army Corps of Engineers at Jackson, Miss., the average ratio of slump to penetration of 1.8 was reported as compared with 1.5 and 1.6 obtained in this study. Walker and Bloem [3] in an unpublished report gave the average ratio of slump to penetration of 1.66 for over 250 tests.

Table 2.—Correlation of Kelly ball penetration test and slump test for consisteney of concrete under laboratory conditions

Identification	Fixed water content [1]		Varying water content [2]			
			Mix No. 1, stone		Mix No. 2, gravel	
	Kelly ball penetration	Slump	Kelly ball penetration	Slump	Kelly ball penetration	Slump
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
A	1.4		0.4	0.6	0.9	1.3
B	1.5		.6	1.2	1.3	1.8
C	1.5		1.3	1.7	1.5	2.5
D	1.7	2.4	1.5	1.9	2.3	3.3
E	1.7	2.5	1.8	3.0	2.6	4.0
F	1.7	3.0	2.2	3.6	2.9	5.0
G	1.7	3.1	2.8	5.0	3.8	5.6
H	1.7				4.5	6.5
I	1.7					
J	2.0					
K	2.2					
Average [3]	1.7	2.8	1.5	2.4	2.5	3.8

[1] Mixture used: 1½-inch maximum size crushed stone, and 6 sacks of non-air-entrained concrete per cubic yard. Each value represents one test.

[2] Mixture used: 1½-inch maximum size coarse aggregate, and 6 sacks of non-air-entrained concrete per cubic yard. Each value for Kelly ball penetration is an average of 6 tests and each value for slump is an average of 2 tests.

[3] Ratios of penetration to slump: for fixed water content, 1:1.6; for varying water content, 1:1.6 for stone and 1:1.5 for gravel.

Advantages of the Kelly Ball Test

On any particular project using specific materials, a limited number of tests will correlate the Kelly ball readings with the corresponding slump tests sufficiently to permit using the Kelly ball for the control of the consistency of the concrete when a slump range has been specified.

The following comments are made on the Kelly ball test as a replacement for the slump test for measuring the consistency and uniformity of concrete in the field:

1. The concrete may be tested in place, therefore the selection or preparation of a sample is eliminated.

2. Three or more Kelly ball tests can be made at a selected location in less time and with less effort than is required for one slump test. Due to the speed with which the test can be made, the operator can work where the concrete is being discharged from the mixer without delaying paving or finishing operations.

3. Making the consisteney test easier and faster should encourage more frequent testing and should be helpful in controlling the uniformity of the concrete.

[3] Stanton Walker and D. L. Bloem, Director of Engineering and Assistant Director of Engineering, respectively, National Sand and Gravel Association and National Ready Mixed Concrete Association.

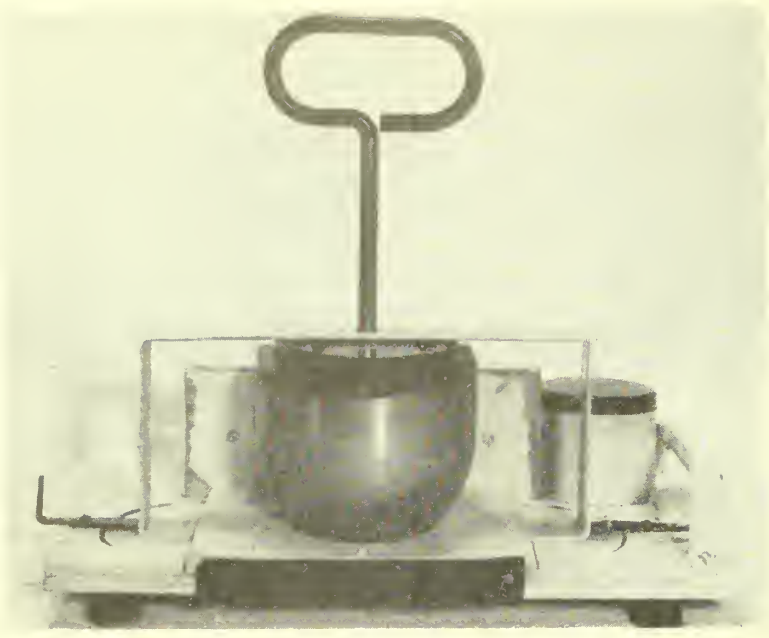

Figure 5.—Field carrying kit for Kelly ball apparatus.

4. The apparatus can be maintained in usable condition between tests by merely wiping with an oily rag.

5. The slump test is not practical for use in testing concrete with a maximum size of coarse aggregate over 2 inches. The Kelly ball penetration test may be used on concrete containing larger aggregate if a sufficient volume is available to provide adequate depth.

Field Kit

For ease in transporting, the ball with wood float and a base plate can be readily assembled into a compact field kit as shown in figure 5.

The wooden float is used to level the concrete at the area to be tested. A tin "rag can" provides a place to keep oily cloth or waste to wipe the ball clean after each test. The ball should not be placed too near the side forms or the edge of a pile of concrete. In order to meet this requirement, the operator frequently must place one foot into the wet concrete. The carrying base plate is designed as a footboard to support the weight of the operator on the plastic concrete.

The apparatus may be built in any machine shop. However, it has been adopted as a tentative standard by the American Society for Testing Materials and may be offered for sale by the leading instrument companies.

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An Improved Sulfate Soundness Test for Aggregates

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by DONALD O. WOOLF
Physical Research Engineer

The present sulfate soundness test method, AASHO Method T 104–46 or ASTM Method C 88–46T, permits in tests of coarse aggregate the determination of six different values any one of which may be considered as "standard." Different laboratories may not agree on the method to be used in testing a given sample of aggregate. An unwarranted dependence is placed on the results of the test. To correct this practice, it is recommended that the method be used for the acceptance but not the rejection of aggregates, that the sample tested be restricted to one size of coarse or fine aggregate, that the drying oven meet requirements for performance, that the loss for coarse aggregate be determined with sieves having openings one-half the size of those in the original retaining sieves, and that certain minor changes be made. It is believed that these changes will result in better agreement between laboratories conducting check tests, and that the test result will permit a reliable appraisal of the soundness of the material under test.

ONE of the first descriptions in American roadbuilding literature of the sodium sulfate soundness test for aggregate appeared in Bulletin 1216 of the U. S. Department of Agriculture. In this bulletin, the tentative standard methods of sampling and testing highway materials were described as adopted in 1922 by the American Association of State Highway Officials. The description of the soundness test is remarkable in its brevity and clearness. It reads as follows:

> Immerse 10 small pieces (total weight about 1,000 grams) of the rock in a saturated solution at 70° F. of sodium sulfate (Na_2SO_4) for 20 hours, after which place them for 4 hours in a drying oven maintained at 100° C. Repeat the treatment five times. Note the condition of the rock as to soundness at the end of the test. Samples which exhibit marked checking, cracking, or disintegration shall be considered to have failed in this test.

With this publication, a long program of study of the sodium sulfate soundness test began. Change after change was made in the method with a view toward broadening it to cover both fine and coarse aggregate, and to standardize it in all its details. In addition, some users saw in the method a chance to develop an acceptance test of aggregate which could be made in a relatively short time in comparison with that required for a freezing and thawing test. With little regard to the actual processes involved in the crystallization of sulfate salts, or of the proper interpretation of the test results, the use of the test for the acceptance or rejection of aggregates was solemnly indorsed. Under pressure for a quick decision, the highway testing engineer turned to the sulfate soundness test as a measure of the acceptability of aggregates. With further misunderstanding of the applicability of the method, requirements for the loss in the sulfate soundness test were written in specifications for aggregates. The use of the method and of requirements based thereon was widely adopted by construction authorities.

Laboratories found difficulty in obtaining reproducible test results, or in testing materials in strict accordance with the requirements of the method. An attempt was made to adjust the method to permit the testing of aggregates of various gradations.

Recommendations

With a view toward securing a dependable, reproducible test method which can be used for a rapid appraisal of the soundness of aggregates, it is recommended that the following changes be made in AASHO Method T 104–46 or the similar ASTM Method C 88–46T:

1. Revise the scope to indicate that the method is indicatory only; that it may be used for acceptance of material but that rejection should be based on other determinations such as freezing and thawing tests, inspection of ledges at the quarry (if the material is rock), or inspection of concrete prepared with the material.

2. Revise the requirements for the oven by the addition of performance determinations.

3. Revise the requirements for the samples to include only the ½-inch to ⅜-inch size of coarse aggregate, and the No. 8 to No. 16 size of fine aggregate.

4. Determine the loss for coarse aggregates with the half-size (No. 4) sieve, but continue the use of the original retaining sieve for fine aggregates.

5. Revise the requirements for the maintenance of the solution to emphasize the necessity for thorough stirring and the breaking of caked salt.

6. Revise the requirements for sample containers to permit the use of perforated or nonperforated containers.

To accompany these changes in the method of test, a change from 12 to 7 percent should be made in the requirements for the loss in the sodium sulfate soundness test given in AASHO Specification M 80 or ASTM Specification C 33 for coarse aggregate for concrete. A similar change for fine aggregate in ASTM Specification C 33 or AASHO Specification M 6 should be made, increasing the allowable loss from 10 to 13.5 percent.

Discussion of Earlier Tests

The first mention of the testing of graded coarse aggregate appeared in the 1931 Proceedings of the American Society for Testing Materials as Method C 89–31T.[1] This method required that the aggregate be separated and tested in the following sizes:

Size	*Minimum weight of sample, grams*
No. 4 (0.185 in.) to ⅜ in	100
⅜ in. to ¾ in	300
¾ in. to 1½ in	1, 500
1½ in. to 2½ in	3, 000
Larger sizes by 1-inch spread	3, 000

The reason for the selection of these sizes is not known, but it appears that the use of the so-called fineness modulus sieves was considered desirable. However, the largest size does not include the 3-inch sieve as used in the fineness modulus series of sieves. It is possible that a difference of one inch between the sizes of the two sieves used to prepare the test sample was as great as was deemed advisable.

Difficulties in obtaining check results of soundness tests between laboratories or even in the same laboratory were the general rule. Possibly in an attempt to correct this, the grading of the sample was more closely defined in AASHO Method T 104–42 and ASTM Method C 88–41T. In these revised methods,

[1] Methods C 88 for fine aggregate and C 89 for coarse aggregate were combined in 1937 and identified as C 88. AASHO methods T 75 and T 76 for the sulfate soundness test of fine and coarse aggregates, respectively, were combined in 1938 and identified as T 104.

the same sizes of coarse aggregate as shown in the 1931 ASTM method were used, but each of the three sizes between the ⅜- and 2½-inch sieves was separated into a coarse and a fine fraction, and these fractions recombined in a prescribed ratio. Some of the minimum weights for sizes tested were also increased. The requirements for the composition of the sample tested are shown in table 1.

The sizings established in this revision did not prove satisfactory. Many aggregates for concrete did not contain material larger than 2 inches, but did contain enough of the 1½- to 2-inch size to require that this size be tested. Considerable confusion developed regarding the sample to be prepared for test. Some laboratories ignored the requirement that the sample contain some 2- to 2½-inch material. Since it was not present in the aggregate as submitted, it could not be tested. These laboratories then tested the 1½- to 2-inch size. Other laboratories adhered more closely to the requirements of the method, and did not test the 1½- to 2-inch size if there was no aggregate larger than 2 inches. The results of check tests between laboratories were not in good agreement and the method was repeatedly condemned as unsatisfactory.

Table 1.—Requirements for sample of aggregate prepared for sulfate soundness test

Size of fragment	Grading 1	Alternate A	Alternate B
Inches	*Grams*	*Grams*	*Grams*
No. 4–⅜	300	300	300
⅜–½	[1] 1,000		500
½–¾		[1] 1,500	750
¾–1	[1] 1,500		1,000
1–1½		[2] 3,000	1,500
1½–2	[2] 3,000		2,000
2–2½			
Larger than specified by 1-inch spread	3,000	3,000	3,000

[1] 33 percent of smaller size and 67 percent of larger size.
[2] 50 percent of each size.

Use of Alternate Gradings

To alleviate some of the troubles found, a second grading for the preparation of the sample was added to the method in 1946 by both the AASHO and the ASTM. This grading, described as Alternate A, required the combination of sizes shown in table 1.

At the same time, a third grading was adopted, possibly to satisfy criticism of the use of the two other combinations of sizes. This grading was identified as Alternate B, and required use of the sizes and weights of sample shown in table 1.

All three of the gradings for the test sample are now equally standardized with respect to this method of test, and confusion may develop if one is designated here as the standard. As two of the gradings are now described as Alternates A and B, the other which was the first grading adopted is identified here as Grading 1.

Determination of Loss

Provision for the determination of the loss by either of two methods can be found in the 1941 and subsequent revisions of the sulfate soundness test. The loss may be measured as the material passing the sieve on which the sample was originally retained, or the material passing a sieve having openings one-half of that size. The initial retaining sieve considered here is that for the combination of sizes of fragment which are tested together. In Grading 1, for example, the original retaining sieve for the ¾- to 1-inch and the 1- to 1½-inch sizes would be the ¾-inch sieve, and the half-size sieve would be the ⅜-inch sieve.

The use of the half-size sieve to determine the loss in the test has three excellent features. It should permit a nominal variation between sieves used in different laboratories to prepare the test sample, and will not require an excessively meticulous separation by size and minute attention to particles of flat or elongated shape. Secondly, it should prevent the accidental inclusion in the loss of pieces which are structurally sound, but due to handling during the test, may suffer minor flaking or chipping of corners and edges. At the completion of the test, these pieces may pass the initial retaining sieve and be included, quite incorrectly, in the loss. Finally, the material passing this half-size sieve would be material which had suffered multiple cracking or disintegration. It would be material which actually was unsound, instead of material which had suffered minor surface damage or had broken along incipient fractures developed in the crushing and handling of the aggregate at the plant.

Variation in Loss Due to Testing Procedure

The method of test for coarse aggregate as now written permits the use of three different gradings, and the determination of the loss by either of two methods. If a sample of certain but not unusual size is tested, there may be some doubt, as previously mentioned, in the sizes to be tested. An aggregate of 2-inch to No. 4 size, containing 5 percent or more of the 2- to 1½-inch size, would be included in this category. If it is desired to obtain the maximum information of the quality of the sample, many laboratories may prepare the test material in accordance with Grading 1, using presumably a 3,000-gram sample of 2- to 1½-inch aggregate. A summary of the test samples which might be prepared with this material is given in table 2.

It is quite unusual for different sizes of a sample of aggregate to have the same or nearly the same loss in the sulfate soundness test. Consequently samples of one aggregate prepared with the three different gradings may have markedly different losses. The loss as determined with the half-size sieve is only in rare cases of the same order of magnitude as that found with the original retaining sieve. Acceptance or rejection of an aggregate may then be governed by the grading used in the test, and the method of determining the loss.

An illustration of this is presented in tables 3–6. The values given in table 3 were obtained in a routine test of a sample of gravel. The grading of the sample as received conformed to AASHO Specification M 80–49 for coarse aggregate. This grading is such that the sample tested may be prepared to have either of the combinations of sizes described here as Grading 1 and Alternate A, or the sample may be tested by the individual size method identified as Alternate B.

Alternate B grading

The sample was tested by the individual size method (Alternate B) to obtain the most information regarding its quality. At the completion of the fifth immersion period, the sample was dried, washed free of the sulfate salt, again dried, and each separate size sieved to determine the losses given in table 3. An average loss for the sample, weighted with respect to the grading of the material as received, was then computed from the losses passing the original retaining (full-size) sieves, and from the losses passing the half-size sieves. These values are given in table 4. Average losses were also computed as if the sample had been tested using Grading 1, or using the Alternate A grading.

Grading 1

The computation of the average weighted loss for Grading 1 is shown in detail in table 5.

Table 2.—Requirements for sample of 2-inch to No. 4 aggregate prepared for sulfate soundness test (AASHO Method T 104–46)

Size of fragment	Grading 1		Alternate A		Alternate B	
	Minimum weight	Sieve used for loss [1]	Minimum weight	Sieve used for loss [1]	Minimum weight	Sieve used for loss [1]
Inches	*Grams*	*Inches*	*Grams*	*Inches*	*Grams*	*Inches*
2–1½	3,000	1½	[2] 3,000	1	2,000	1½
1½–1	[3] 1,500	¾			1,500	1
1–¾			[3] 1,500	½	1,000	¾
¾–½	[3] 1,000	⅜			750	½
½–⅜			[4] 300	No. 4	500	⅜
⅜–No. 4	300	No. 4			300	No. 4
Total minimum weight	5,800		4,800		6,050	

[1] A sieve having an opening one-half of the size given may be used as an alternate.
[2] 50 percent of each size.
[3] 67 percent of larger size and 33 percent of smaller size.
[4] Applies to ½-inch to No. 4 aggregate; no requirements given for sizes separated by the ⅜-inch sieve.

Table 3.—Grading of sample and loss in sulfate soundness test

Size tested	Grading as received	Loss: Amount passing sieve after test as a percentage of fraction originally retained on each sieve							
		1½ in.	1 in.	¾ in.	½ in.	⅜ in.	No. 3	No. 4	No. 8
Inches	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2–1½	5	19.4	10.6	4.8	2.3	----	------	------	-----
1½–1	21	------	11.8	11.2	10.9	9.5	------	------	-----
1–¾	19	------	------	17.2	12.0	8.1	5.0	------	-----
¾–½	17	------	------	------	18.1	13.1	8.9	5.4	-----
½–⅜	16	------	------	------	------	17.0	11.8	8.8	6.8
⅜–No. 4	22	------	------	------	------	------	------	11.3	7.0

This grading requires that the 1½- to 1-inch and 1- to ¾-inch sizes be tested together as well as the ¾- to ½-inch and ½- to ⅜-inch sizes. In each case, the tested material contains 67 percent of the larger size and 33 percent of the smaller size. The original retaining sieve for the combination of the 1½- to 1-inch and 1- to ¾-inch sizes is considered here to be the ¾-inch sieve, and a half-size sieve would be the ⅜-inch sieve. Similarily, the ⅜-inch and No. 4 sieves are considered to be the original retaining and half-size sieves, respectively, for the combination of the ¾- to ½-inch and ½- to ⅜-inch sizes.

The effect of using these combined sizes can be obtained by the use of conversion factors as shown in table 5. The actual loss for the 1½- to 1-inch size passing the ¾-inch sieve is multiplied by 0.67 and added to 0.33 times the actual loss for the 1- to ¾-inch size passing the same sieve. The actual losses passing the ⅜-inch sieve for the ¾- to ½-inch, and ½- to ⅜-inch sizes are similarly combined. The losses for these combined sizes as well as those for the two sizes which were not combined, are used to compute the weighted average loss.

Alternate A grading

Table 6 shows the computation of the weighted average for the sample as if it had been tested using the Alternate A grading. The 2- to 1½-inch and 1½- to 1-inch sizes are combined in equal weights for testing, but the 1- to ¾-inch and ¾- to ½-inch sizes are combined in the 67–33 ratio. In the table, appropriate conversion factors are used. The Alternate A grading does not show any separation of the ½-inch to No. 4 size. As this size was separated on the ⅜-inch sieve and so tested, the losses for the ½- to ⅜-inch and ⅜-inch to No. 4 sizes have been combined in the same ratio that these sizes occurred in the original sample.

The results given in tables 4–6 are summarized in table 7. If the sample is tested by the Grading 1 method and the loss considered as the material passing the initial retaining sieve, the material will not meet the the requirement of 12 percent given in ASTM Specification C 33–52T or in AASHO Specification M 80–51 for coarse aggregate for portland cement concrete. If the sample is tested with the grading given in the specification as Alternate A, the loss in the test will meet the requirements of the specification. Should the sample be tested using the individual sizes permitted under Alternate B, it would fail to meet the requirement for soundness. If the loss is determined, as permitted, with half-size sieves, the material could be reported as of acceptable quality. Since almost all

Table 4.—Computations of test results for sample tested with Alternate B grading

Size tested (inches)	Actual loss (percent)	Grading as received (percent)	Weighted average loss (percent)
Loss Determined Using Full-Size Sieves			
2–1½	19.4	5	0.97
1½–1	11.8	21	2.48
1–¾	17.2	19	3.27
¾–½	18.1	17	3.08
½–⅜	17.0	16	2.72
⅜–No. 4	11.3	22	2.48
Total	----	100	15.0
Loss Determined Using Half-Size Sieves			
2–1½	4.8	5	0.24
1½–1	10.9	21	2.29
1–¾	8.1	19	1.54
¾–½	8.9	17	1.51
½–⅜	8.8	16	1.41
⅜–No. 4	7.0	22	1.54
Total	----	100	8.5

Table 5.—Computation of test results for sample tested with Grading 1

Size tested	Actual loss	Conversion factor	Converted values for loss	Loss for specified combinations of sizes	Grading as received	Weighted average loss
Inches	*Pct.*		*Pct.*	*Pct.*	*Pct.*	*Pct.*
Loss Determined Using Full-Size Sieves						
2–1½	19.4	1.00	19.4	19.4	5	0.97
1½–1	11.2	.67	7.5	13.2	40	5.28
1–¾	17.2	.33	5.7			
¾–½	13.1	.67	8.8	14.4	33	4.75
½–⅜	17.0	.33	5.6			
⅜–No. 4	11.3	1.00	11.3	11.3	22	2.49
Total	----	----	----	----	100	13.5
Loss Determined Using Half-Size Sieves						
2–1½	4.8	1.00	4.8	4.8	5	0.24
1½–1	9.5	.67	6.4	9.1	40	3.64
1–¾	8.1	.33	2.7			
¾–½	5.4	.67	3.6	6.5	33	2.14
½–⅜	8.8	.33	2.9			
⅜–No. 4	7.0	1.00	7.0	7.0	22	1.54
Total	----	----	----	----	100	7.6

Table 6.—Computations of test results for sample tested with Alternate A grading

Size tested	Actual loss	Conversion factor	Converted values for loss	Loss for specified combinations of sizes	Grading as received	Weighted average loss
Inches	*Percent*		*Percent*	*Percent*	*Percent*	*Percent*
Loss Determined Using Full-Size Sieves						
2–1½	10.6	0.50	5.3	11.2	26	2.91
1½–1	11.8	.50	5.9			
1–¾	12.0	.67	8.0	14.0	36	5.04
¾–½	18.1	.33	6.0			
½–⅜	8.8	[1].42	3.7	10.2	38	3.87
⅜–No. 4	11.3	[1].58	6.5			
Total	----	----	----	----	100	11.8
Loss Determined Using Half-Size Sieves						
2–1½	2.3	0.50	1.2	6.6	26	1.72
1½–1	10.9	.50	5.4			
1–¾	5.0	.67	3.4	6.3	36	2.27
¾–½	8.9	.33	2.9			
½–⅜	6.8	[1].42	2.9	7.0	38	2.66
⅜–No. 4	7.0	[1].58	4.1			
Total	----	----	----	----	100	6.7

[1] Relative amounts of these sizes to be used are not shown in the method of test. Conversion factors used are based on amount of each size in material as received.

Table 7.—Summary of test results for samples tested with three methods of grading

Method	Loss with full-size sieve	Loss with half-size sieve
	Percent	*Percent*
Grading I	13.5	7.6
Alternate A	11.8	6.7
Alternate B	15.0	8.5

specifications for aggregate fail to mention which procedure of those available shall be used, it is possible that the acceptance or rejection of a given sample may depend entirely on which one of the six standard methods of test has been used.

Soundness Questionnaire

Reports of cooperative tests made using the 1946 revision of the method indicated that different laboratories failed to obtain a reasonable agreement in the results of tests on the same material. In an effort to determine the cause for this, Subcommittee III-c of Committee C–9, ASTM, sent a questionnaire covering the sulfate soundness test to all laboratories known to use the test. The questionnaire covered in detail the apparatus used and the procedures followed by each laboratory. About 50 laboratories returned the questionnaire. Of these only three stated that all details of the method were followed exactly. Many laboratories indicated departures from the requirements of the specifications in matters which were considered immaterial. Others showed variations which admittedly could influence the test result but which were necessitated by one reason or another including shortage of help in the laboratory. In some respects, the inference could be drawn from replies to the questionnaire that some requirements of the method were too strict, or were unnecessary, and that the operator would conform to these only under protest.

The questionnaire furnished information concerning necessary or desirable changes in the requirements for the method. It was disclosed that many laboratories use pans instead of perforated containers for holding the test samples. Although the use of pans is not permitted in the present method, the only advantage of the use of perforated containers is that the sample can be drained more thoroughly and dried more rapidly. If laboratories have found that samples stored in pans can be brought to oven dryness in a suitable time, and it is presumed that this is so, the requirements of the method should be changed to accept this established practice. A test for the rate of evaporation of the drying ovens was suggested by the writer and furnished some surprising results. The present specifications require that the oven "shall provide a free circulation of air through the oven and shall be capable of maintaining a temperature of 105 to 110° C." The questionnaire requested that a 1,000 ml. beaker containing 500 grams of water at 70° F. be placed in the oven with a group of test samples to be dried, and that the amount of water evaporated in a given time be determined.

Evaporation rate of ovens

Information regarding the rate of evaporation of the oven was furnished by 38 laboratories. The values reported were reduced to grams of water evaporated per hour, and plotted in figure 1 as a frequency distribution curve. The average rate was about 28 grams per hour under the established conditions. The maximum and minimum rates were 61.5 and 1.5 grams per hour. Twenty percent of the ovens had a rate of 22 grams per hour or less, and an equal number had a rate of 45 grams per hour or more. Of 11 ovens of the mechanical convection type, 8 were reported to have an evaporation rate of 28 grams per hour or more.

Significant differences were found between the work done by the various ovens on the basis of the evaporation rate and the reported time normally used for the drying cycle. The computed values for the total amount of water evaporated, assuming the rate remained constant, varied from 16 to 1,080 grams. Only 33 laboratories furnished definite statements of the time normally used during the drying cycle, and the ovens involved are rated as follows:

Grams of water evaporated during drying cycle	*Number of ovens*
Less than 50	1
50–100	4
100–150	10
150–200	3
200–300	8
300–400	3
Over 400	4

It is entirely possible that some of the information from which these values were computed was reported incorrectly. If so, the great differences shown between ovens may be reduced. There were, however, sufficient differences in the rate of evaporation of the various ovens to warrant including in the method, requirements for the performance of the oven. A suitable revision of the requirements for the oven follows. The values included are based on the performance of the ovens reported in the questionnaire, and may need some revision when a larger number of ovens are studied.

> The oven shall be capable of being heated continuously between 105 and 110° C. (221 and 230° F.) and the rate of evaporation of water shall be at least 25 grams per hour. This rate shall be determined by the loss of water from 1-liter, Griffin low form beakers, each containing 500 grams of water at 70° F., placed at each corner and the center of each shelf of the preheated oven, and heated for at least 4 hours during which period the doors of the oven shall be kept closed. The rate of evaporation shall be determined from the average loss for all beakers.

Consideration had been given to the exclusive use of magnesium sulfate in this method because the solubility of this salt is much less variable with change in temperature than that of the sodium salt. With the smaller variation in solubility, a saturated solution of the magnesium salt could more readily be maintained, and duplication of test results should be more certain. In the replies to the questionnaire, it was found that of 33 laboratories which used the method regularly, 21 used sodium sulfate exclusively or as an alternate with the magnesium salt. The fact that almost two-thirds of the reporting laboratories

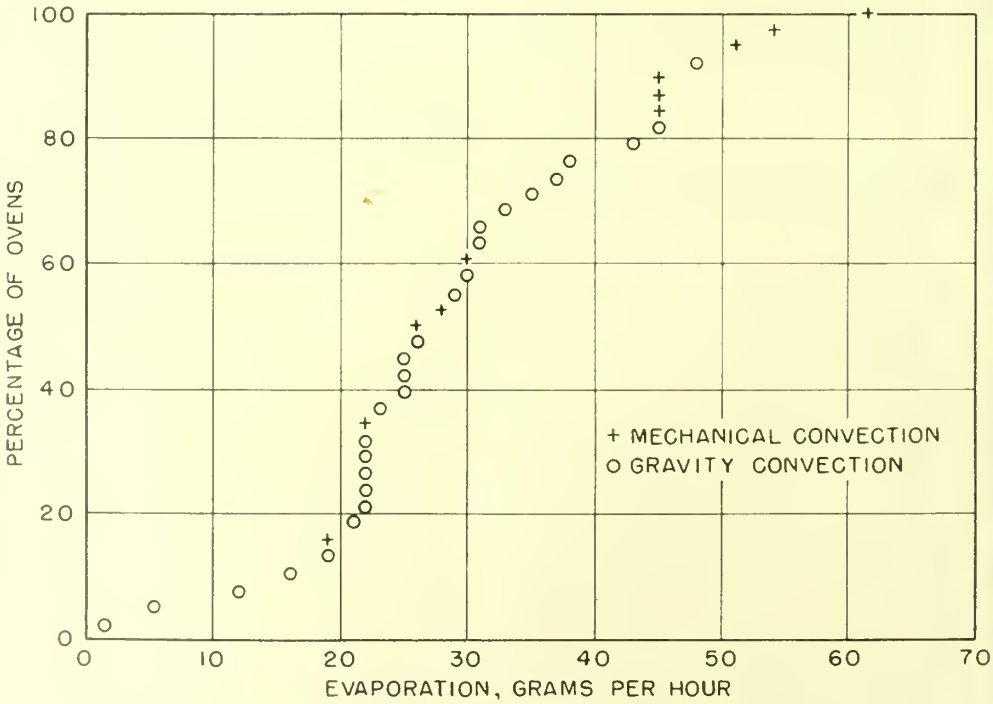

Figure 1.—Rate of evaporation of 38 ovens used in accelerated soundness test.

used sodium sulfate makes it inadvisable to consider its elimination at this time.

Basis for rejection of aggregate

Replies to the questionnaire were received from 23 laboratories which regularly make the test on concrete aggregates proposed for use in construction financed by public funds. Eleven of these laboratories stated that material is rejected on the basis of the sulfate soundness test. The other 12 stated that material is not so rejected, but that the results of freezing and thawing or other tests, with or without service records, are considered. The use of the sulfate soundness test for the arbitrary rejection of aggregate is considered a highly questionable practice. Experience has shown that materials which have a low loss in the test generally have adequate resistance to frost action. It has also been shown that many materials having a high loss in the sulfate soundness test may not resist freezing and thawing, but this is not true of all materials.

A low loss in the soundness test can be associated with resistance to weathering with more assurance than a high loss and lack of resistance to weathering. As an example, a sample of sandstone had a loss of 64 percent in a five-cycle sodium sulfate test. Another portion of the same sample showed a loss of only 2 percent when frozen and thawed 50 times in water. None of the fragments cracked or broke in freezing, and some or all of the loss may have been due to handling. As the stone in question is widely used for building with satisfactory results, it must be concluded that the results of the sulfate soundness test were quite misleading in this case.

It is considered highly advisable to add to the method a definite statement of the applicability of the test result. A suitable revision of the "Scope" of the method could be made by the insertion of one sentence (given in italic) to have the text read as follows:

> This method covers the procedure to be followed in testing aggregates to determine their resistance to disintegration by saturated solutions of sodium sulfate or magnesium sulfate. It furnishes information helpful in judging the soundness of aggregates subject to weathering action, particularly when adequate information is not available from service records of the material exposed to actual weathering conditions. *Generally the results of this test should not be used to reject materials, but aggregates having a low loss in this test can confidently be assumed to be resistant to the effects of freezing and thawing.* Attention is called to the fact that test results by the use of the two salts differ considerably and care must be exercised in fixing proper limits in any specifications which may include requirements for these tests.

Sulfate solution

A number of replies to the questionnaire called attention to two other items which unquestionably have a marked effect on the test result. These are the preparation and maintenance of the sulfate solution. The method requires that a saturated solution of the salt be prepared, and gives quite detailed instructions regarding the procedure to be followed. This does not appear to be a formidable undertaking, nor does it appear difficult to maintain that solution in its saturated condition provided the temperature of the solution can be controlled within the specified limits of 68 to 72° F.

Comment was made that the present requirements in the method call for an excessive amount of the sulfate. Use of the amount indicated was found to furnish a solid crystalline mass of the sulfate in the container for the saturated solution. As the solid crystal does not enter solution readily, it is quite possible for the liquid in the container to become less than saturated even in the presence of this crystalline mass. The method of test now requires that the solution shall be stirred thoroughly immediately before use. It is doubted that this will cause material from the crystalline mass to enter solution unless the mass is broken to fine size. Consequently it is recommended that the method be revised to require breaking of the mass prior to agitation. The addition of a suitable statement to the note concerning the amount of salt to be used appears desirable.

Half-Size Sieves and the Reverse Cycle

The use of a sieve having openings one-half the size of those in the sieve used to prepare the sample has already been mentioned. The loss through the half-size sieve should be a better indication of the soundness of a material than that passing the initial retaining sieve. When the latter sieve is used, some otherwise sound fragments may pass the sieve due to minor chipping or breaking of corners or edges. Although these chips would probably pass a half-size sieve, any considerable amount of material passing this sieve would be truly unsound, having suffered multiple cracking or marked disintegration. With elimination of the accidental inclusion of sound fragments in the loss, more concordant results between laboratories should be obtained.

In the present method, the sample is immersed in the sulfate solution for 16 to 18 hours, and then dried to constant weight. Most laboratories endeavor to maintain a 24-hour cycle, and limit the drying period to 5 to 7 hours. As it has been shown that this period of drying may not be sufficient to dry the sample thoroughly, consideration was given to the use of a reverse cycle method. In this, the sample was immersed in the sulfate solution for about 6 hours, and dried in the oven for 16 to 17 hours. Previous work had indicated that coarse aggregate would absorb as much water in 6 hours as it would in 24 hours, and that a drying period of the length shown would dry soundness samples even in ovens with a relatively low rate of evaporation. As thorough drying of the sample is necessary for the maximum crystallization of the sulfate salt, it was believed that the most effective results and the desired 24-hour cycle could be obtained with the reverse cycle method.

Tests of coarse aggregate were made to determine whether the use of half-size sieves or the reverse cycle would furnish more concordant results. Nine samples of stone of high to low loss were selected to determine the suitability of the procedures mentioned above. Each sample was crushed and screened to the 1½- to 1-inch size. Six test samples were prepared from each stone to make a total of 54 samples tested. Three samples of each material were subjected to five repetitions of the standard cycle consisting of an immersion period of 17 hours and a drying period of 6 hours. An equal number of sam-

Table 8.—Tests by standard and reverse cycle methods on 1½- to 1-inch stone

Sample number	Loss passing full-size sieve				Loss passing half-size sieve			
	Test 1	Test 2	Test 3	Average	Test 1	Test 2	Test 3	Average
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
STANDARD CYCLE								
89805	45.0	39.2	45.6	43.3	20.5	16.9	15.9	17.8
89812	19.9	32.4	8.1	20.1	8.7	11.6	3.0	7.8
89813	17.2	19.5	33.3	23.3	6.8	5.6	8.2	6.9
89889	8.0	6.2	2.0	5.4	4.5	4.5	1.4	3.5
89890	74.0	84.2	77.2	78.5	51.7	55.6	57.5	54.9
89892	10.1	4.2	4.1	6.1	3.9	1.5	3.6	3.0
89916	13.4	6.0	14.1	11.2	3.8	.5	4.8	3.0
89918	14.2	7.0	32.7	18.0	6.5	2.1	16.5	8.4
89920	40.2	37.5	36.7	38.1	7.1	8.8	12.6	9.5
REVERSE CYCLE								
89805	41.9	28.7	56.4	42.3	17.3	9.3	23.2	16.6
89812	7.3	15.1	4.0	8.8	2.3	5.3	2.0	3.2
89813	12.5	17.7	22.2	17.5	5.4	6.4	9.8	7.2
89889	2.8	4.3	4.6	3.9	2.8	2.6	4.1	3.2
89890	85.3	53.4	94.5	77.7	55.1	34.6	68.9	52.9
89892	12.7	6.1	12.9	10.6	5.4	3.6	4.7	4.6
89916	.8	6.5	8.6	5.3	.1	1.9	2.2	1.4
89918	19.6	11.0	14.5	15.0	9.8	6.3	6.2	7.4
89920	18.5	8.8	19.2	15.5	4.9	.3	5.5	3.6

Table 9.—Average values and mean deviations from average values for tests by standard and reverse cycle methods

Method	Sieve used to determine loss	Average loss, all samples	Mean deviation from average loss
		Percent	*Percent*
Standard cycle	Full-size	27.1	28.2
Do	Half-size	12.8	31.5
Reverse cycle	Full-size	21.8	29.7
Do	Half-size	11.1	33.0

ples were subjected to the reverse cycle with an immersion period of 6 hours and a drying period of 17 hours. At the completion of the fifth cycle, each sample was washed, dried, and sieved to refusal on the 1- and ½-inch sieves. The results obtained are shown in table 8.

Variation in test results

It will be observed that some test results differ to a marked extent from companion results for the same material. This is found in tests by each method. Computation of a mean deviation from the average for all results obtained by each method of treatment and each method of determining the loss furnished very nearly the same value for each of the four groups of results. This indicates that nonuniformity of the materials tested probably is responsible for the variations found in some of the test results. It is believed that the effect of the wild results will be cancelled out if all results obtained by each of the four methods of test are considered, and that comparisons between average values for each method are valid. These average values as well as values for mean deviations from the average are shown in table 9.

It was expected that the reverse cycle would insure more thorough drying of the samples than would be obtained with the standard method. Due to this, it was believed that the reverse cycle would furnish a greater loss for each material with more uniform results from sample to sample. These were not obtained because the oven used had a sufficient rate of evaporation to dry the samples in the 6-hour period provided in the standard method.

Several matters of interest are found in the results obtained in these tests. The smaller losses for the reverse cycle method indicate that insufficient time was allowed for either the absorption of the sulfate solution or the crystallization of the salt, or both. The better agreement between the average losses determined with the half-size sieve than those for the full-size sieve indicates that with reduction in size of the fragment tested, the time allowed in the reverse cycle for absorption or crystallization becomes less critical.

Absorption rate of sulfate solution

These results are further interpreted to indicate that the factor controlling the loss in the reverse cycle is the time for absorption of the sulfate solution. The rate of crystallization would be the same irrespective of the method used, but the rate of absorption would vary inversely with the size of fragment.

The rate of absorption of the sulfate solution may also be affected if the mouth of the capillary tube or crack in the fragment is plugged by the sulfate salt. The salt carried into the rock during each immersion period tends to be drawn toward the surface of the rock during the drying period and forms a deposit at or near the surface. During subsequent immersion periods, the solution must pass this plug. As the solution is saturated, the plug of salt cannot be dissolved but must be softened until the solution can push past or around it. Considerable time probably is required for this and the 6-hour immersion period in the reverse cycle may be insufficient to permit complete saturation of the larger sizes of coarse aggregate.

It is believed that with the adoption of requirements for the oven used in this method of test, samples may be dried in a period of 6 hours and use of a reverse cycle will be unnecessary. The use of the half-size sieve in the determination of the loss of samples of coarse aggregate is recommended.

Size of Sample

Coarse aggregate

For a coarse aggregate graded from 2 inches to the No. 4 sieve, the present method requires a test sample weighing at least 4,800 to 6,050 grams, depending upon the grading chosen for the material tested. This material is separated into 3 to 6 portions also depending on the grading to be used. If several samples

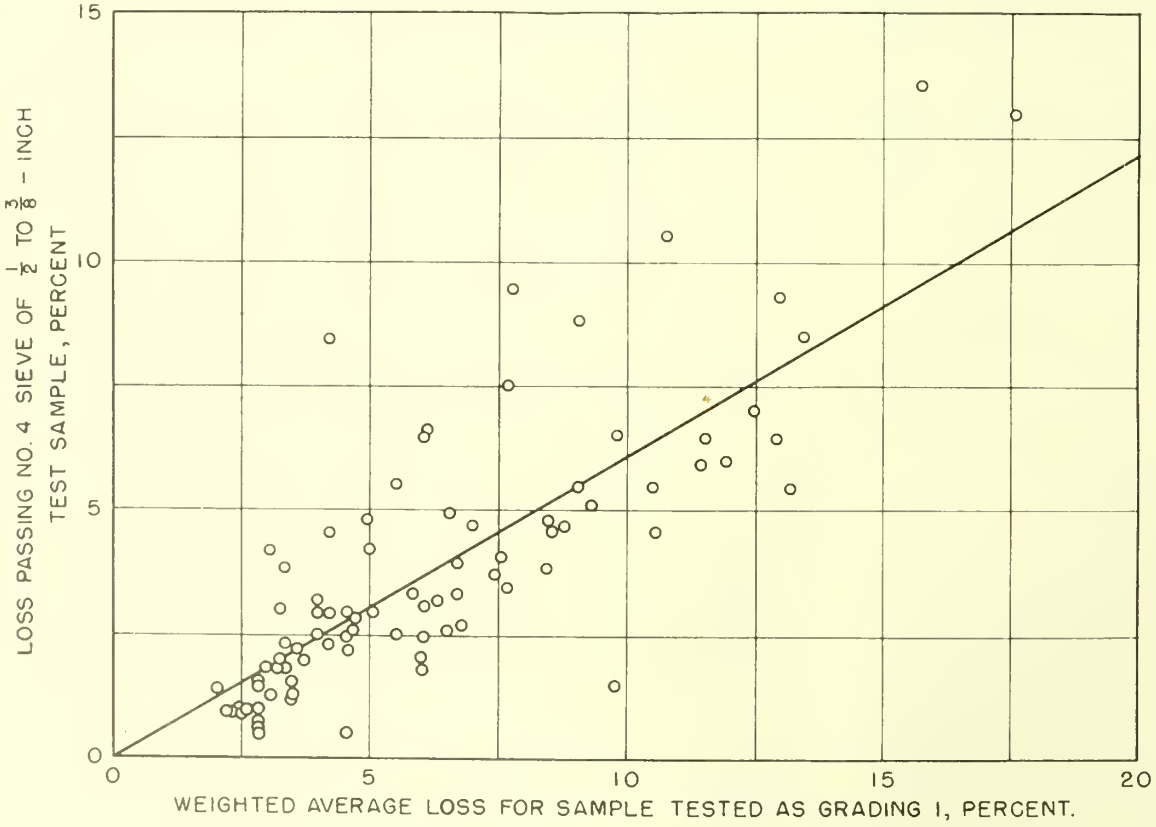

Figure 2.—Comparison of losses for coarse aggregate.

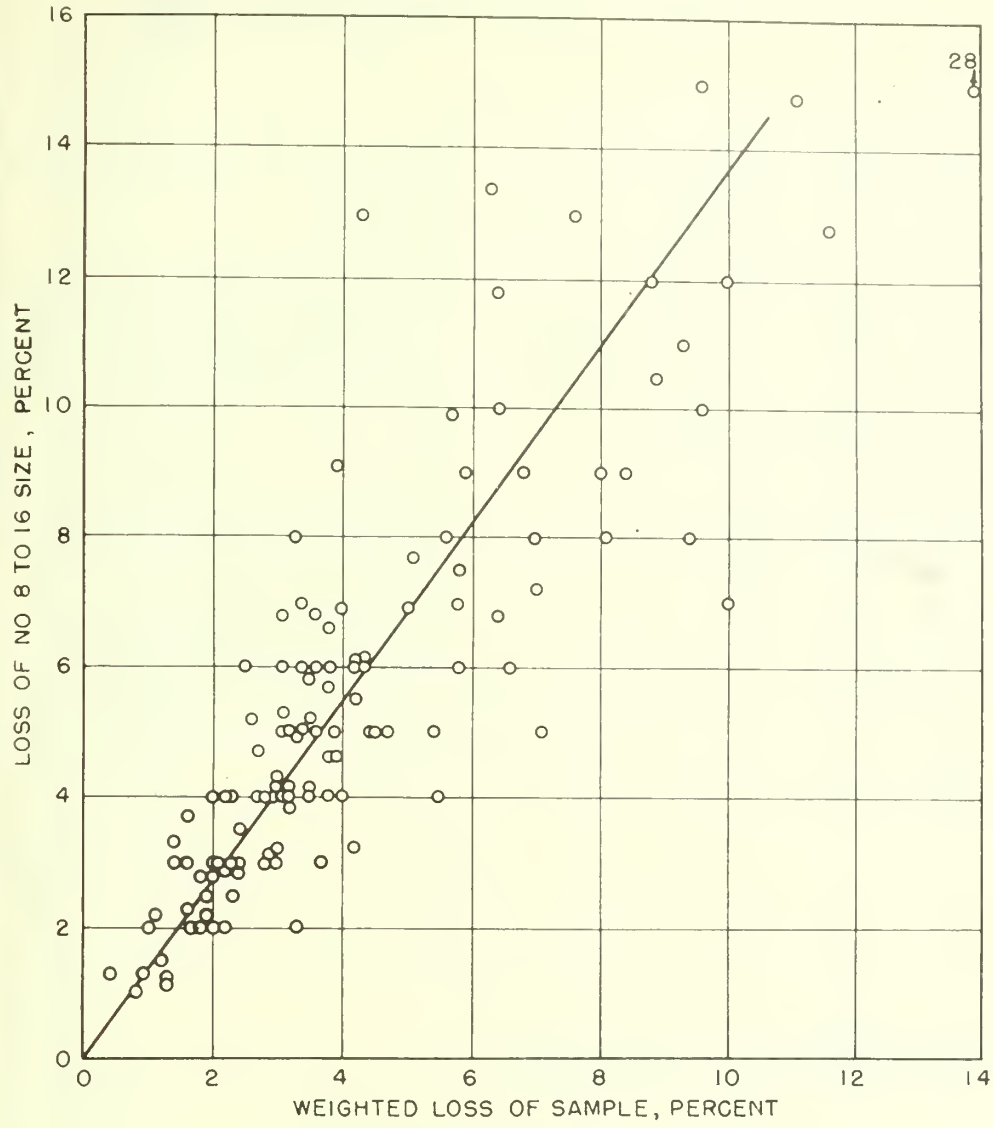

Figure 3.—Comparison of losses for fine aggregate.

are tested simultaneously, the total amount of material under test and the number of containers used may tax the facilities of the laboratory. In laboratories with limited personnel, it has been observed that there is a marked tendency for reduction in the amount of material tested.

Although this reduction may have little or no effect on materials of uniform composition, it might tend to prevent reliable test results on nonhomogeneous materials. To determine whether any change could be made in the method which would permit the use of a smaller sample and reduce the labor and attention required, the results of tests of over 250 samples were studied. These samples had been tested in the laboratory of the Bureau of Public Roads during the period 1945–53.

Most of the samples of coarse aggregate had a maximum size of 1½ or 2 inches, and most had been tested by individual size of fragment, the Alternate B method. The results were studied to determine whether the number of sizes of aggregate could be reduced and still obtain a loss approximating that for all sizes tested. Comparison of the averages for the entire group of samples indicated that the loss for the ½- to ⅜-inch size agreed closely with the loss for all sizes tested. From these data, it would appear that a good knowledge of the soundness of a material may be obtained by test of the ½- to ⅜-inch size alone. As a test of this size would require a test sample weighing only 500 grams, a great reduction in the weight of material and number of samples tested would result.

Losses through both the full-size and half-size sieves had been determined for about 150 of the samples tested by the Alternate B method. To determine the effect on the specification requirement of a desirable change in the method of testing, figure 2 was prepared from data regarding these samples. This shows a comparison between the weighted average loss through the original retaining sieve of samples of 1½- or 2-inch maximum size coarse aggregate when tested as required for Grading 1, and the loss of ½- to ⅜-inch aggregate passing a half-size sieve. Samples which had a weighted average loss of less than 2 percent were not plotted, nor were several samples with quite high losses.

ASTM Specification C 33 and AASHO Specification M 80 permit the acceptance of coarse aggregate for concrete having a loss in the sodium sulfate soundness test of 12 percent. As both specifications are silent regarding the method for determining this loss, the method considered here is that involving the use of Grading 1 with the loss passing the original retaining or full-size sieve. Figure 2 shows that material of the quality indicated will have a loss of 7 percent passing the half-size (No. 4) sieve when tested as ½- to ⅜-inch material. It is believed that sufficient data have been presented to insure the reliability of this conversion in specification requirements.

Fine aggregate

A different problem was found in connection with the most desirable method for testing fine aggregate, especially natural sand. As a general rule, the loss of natural sand in a soundness test varies directly with the size of the sand grain. As the size of grain is reduced, the percentage of single grains of quartz increases, and these grains have very little loss in a soundness test. It is believed that the testing for soundness of fine grains of natural sand is largely wasted effort, and that a large loss found in tests of such material probably resulted from accidental loss of some of the sample.

However, an adequate knowledge of the soundness of fine aggregate can be obtained by test of a single size of grain. The selection of the size tested must be based on the availability of that size in sand as customarily used, and on the occurrence in that size of the more unsound grains. It is believed that the No. 8 to No. 16 size best meets these requirements. Although a coarser size may contain more unsound grains, many sands have very little material retained on the No. 8 sieve, and the use of this coarser size cannot be considered.

A comparison between the average loss weighted with respect to the grading of the sample as required by the present method of test and the loss for the No. 8 to No. 16 size, is shown in figure 3. The loss in each case was determined from material passing the original retaining sieve. Approximately 125 samples of sand are represented. The loss for the single size is slightly greater than the weighted average for all sizes. A sand having an average loss for all sizes of 10 percent would have a loss of about 13.5 percent for the No. 8 to No. 16 size. Established specification limits for fine aggregate can readily be revised to make provision for this difference.

Annual Report of the Bureau of Public Roads, Fiscal Year 1955

The *Annual Report of the Bureau of Public Roads* for fiscal year 1955 is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

Reflecting the prime significance of highway transport, the report covers a wide range of engineering, administrative, and research activities in this field. It discusses all phases of the Federal-aid construction program, which reached new high levels in 1955. Included are significant improvements on the National System of Interstate Highways as well as developments on other primary and urban highways and farm-to-market roads.

To accelerate the highway improvement program, the Federal-aid authorization of $875 million for the fiscal year 1956 was apportioned to the States on July 1, 1954, 6 months ahead of the time limit set by Congress.

During the fiscal year, $671 million of Federal funds were used in the construction of 22,155 miles of highways with a total cost of $1,280 million. Included were 6,050 miles of highways and 1,202 bridges on the Federal-aid primary system outside of cities, 842 miles of highways and 465 bridges on urban portions of the primary system, 14,692 miles of roads and 1,764 bridges on secondary roads, and 571 miles of highways in National parks, forests, parkways, and on flood-relief projects. Railway-highway grade crossings were eliminated at 216 locations, and 317 crossings were protected by installation of appropriate safety devices.

A number of outstanding Federal-aid projects are described in the report, which also reviews factors affecting progress, new highway legislation, foreign activities including improvements on the Inter-American Highway, and work in the field of traffic safety.

Urban problems received particular attention. As the report shows, in the selecting of city projects for Federal aid every effort was made to eliminate traffic bottlenecks by construction that provided greater traffic capacity. Expressways were under construction in more than 100 cities. Construction of circumferential routes near the outskirts of metropolitan areas was started in Boston, Louisville, Baltimore, and other centers. This will permit a free exchange of traffic between radial routes without passing through the more congested areas. Great stress was laid on the need for full control of access on all arterial highways.

Four special reports were submitted to Congress by the Bureau, among them the most comprehensive study of highway needs ever undertaken. This report, prepared in cooperation with the State highway departments, showed that the cost of construction required to modernize the Nation's roads and streets in a period of 10 years would be $101 billion.

In the field of research, the annual report covers Bureau studies of highway finance, highway transport, and the array of physical problems that are associated with highway construction.

Reviewing other significant developments, the report points out that the record amount of highway construction during the year was overshadowed in the public mind by the need for a much greater volume of highway improvements to reduce congestion, accidents, and delays.

The Identification of Rock Types

To meet popular demand, a convenient 6 x 9-inch reprint has again been made of the article *The Identification of Rock Types*, by D. O. Woolf, which appeared in PUBLIC ROADS, vol. 26, No. 2, June 1950. The article presents a simple method for use by the highway engineer in making field identification of the different types of rock with which he is concerned. It is extremely useful to engineers, engineering students, and others whose work requires a limited, practical knowledge of geology. The reprint is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents a copy.

U. S. GOVERNMENT PRINTING OFFICE: 1956

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
1941, 15 cents. 1948, 20 cents.
1942, 10 cents. 1949, 25 cents.

Public Roads Administration Annual Reports:
1943; 1944; 1945; 1946; 1947.
(*Free from Bureau of Public Roads*)

Annual Reports of the Bureau of Public Roads:
1950, 25 cents. 1952, 25 cents. 1954 (out of print).
1951, 35 cents. 1953, 25 cents. 1955, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). 55 cents.

Construction of Private Driveways, No. 272MP (1937). 15 cents.

Criteria for Prestressed Concrete Bridges (1954). 15 cents.

Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.

Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.

Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.

Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Bridge Location No. 1486D (1927). 15 cents.

Highway Capacity Manual (1950). $1.00.

Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.

Highway Practice in the United States of America (1949). 75 cents.

Highway Statistics (annual):
1945 (out of print). 1948, 65 cents. 1951, 60 cents.
1946, 50 cents. 1949, 55 cents. 1952, 75 cents.
1947, 45 cents. 1950 (out of print). 1953, $1.00.

Highway Statistics, Summary to 1945. 40 cents.

Highways in the United States, *nontechnical* (1954). 20 cents.

Highways of History (1939). 25 cents.

Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.

Interregional Highways, House Document No. 379 (1944). 75 cents.

Legal Aspects of Controlling Highway Access (1945). 15 cents.

Local Rural Road Problem (1950). 20 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

Model Traffic Ordinance (revised 1953). Out of print.

PUBLICATIONS (Continued)

Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.

Opportunities in the Bureau of Public Roads for Young Engineers (1955). 25 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.

Public Control of Highway Access and Roadside Development (1947). 35 cents.

Public Land Acquisition for Highway Purposes (1943). 10 cents.

Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

Roadside Improvement, No. 191MP (1934). 10 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP–41 (1948). $1.50.

Standard Plans for Highway Bridge Superstructures (1953). $1.25.

Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.

Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

United States System of Numbered Highways together with the Federal-Aid Highway System (also shows in color National forests, parks, and other reservations). 5 by 7 feet (in 2 sheets), scale 1 inch equals 37 miles. $1.25.

United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24–27.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1955

FOR OFFICIAL DISTRIBUTION

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$11,324	$20,141	$11,126	356.3	$7,579	$4,024	59.8	$35,759	$18,696	447.7	$63,479	$33,846	863.8
Arizona	3,937	9,386	6,957	171.2	2,476	1,896	38.4	9,016	6,587	118.9	20,878	15,440	328.5
Arkansas	14,368	9,878	5,271	311.5	4,375	2,172	45.0	20,962	10,601	414.0	35,215	18,044	770.5
California	11,007	33,812	18,537	201.6	25,025	13,016	23.9	131,906	67,462	276.5	190,743	99,015	502.0
Colorado	17,617	7,489	4,158	90.1	2,827	1,701	25.8	13,662	7,338	148.3	23,978	13,197	264.2
Connecticut	22,659	1,760	985	5.2	366	182	.3	8,261	4,107	12.4	10,387	5,274	17.9
Delaware	6,006	2,079	1,055	9.6	1,607	828	29.1	4,113	2,061	21.7	7,799	3,944	60.4
Florida	11,003	13,096	7,031	219.5	16,626	8,675	76.3	29,561	15,070	307.1	59,283	30,776	602.9
Georgia	23,503	24,294	12,591	434.2	9,441	4,316	68.6	39,108	18,520	697.2	72,843	35,427	1,200.0
Idaho	8,298	4,482	2,965	61.5	2,920	2,070	44.8	11,980	7,716	155.2	19,382	12,751	261.5
Illinois	21,136	54,038	29,365	498.3	15,460	8,326	31.7	82,503	44,397	466.6	152,001	82,088	996.6
Indiana	31,106	16,294	8,349	71.4	18,223	10,148	95.3	36,002	19,244	80.4	70,519	37,741	247.1
Iowa	14,289	20,000	11,326	517.9	8,245	4,253	118.9	18,519	9,858	825.7	46,764	25,437	1,462.5
Kansas	13,781	11,176	5,865	628.1	7,874	4,159	130.5	22,957	11,744	870.1	42,007	21,768	1,628.7
Kentucky	15,335	4,738	2,452	19.0	6,273	3,862	13.4	36,382	18,773	601.2	47,393	25,087	633.6
Louisiana	13,547	11,163	5,582	108.0	4,214	2,107	12.9	45,793	21,777	513.4	61,170	29,466	634.3
Maine	7,993	7,688	4,124	55.9	650	286	1.4	10,048	5,151	68.0	18,386	9,561	125.3
Maryland	8,740	23,603	12,065	58.8	8,437	4,144	8.1	15,900	8,435	64.5	47,940	24,644	131.4
Massachusetts	13,658	23,663	12,501	32.9	7,578	3,767	6.5	46,669	22,429	55.0	77,910	38,697	94.4
Michigan	17,090	40,530	20,976	481.4	23,625	13,183	100.7	43,310	21,516	321.1	107,465	55,675	903.2
Minnesota	15,437	18,907	9,537	576.1	2,389	1,627	73.2	26,795	14,253	476.3	48,091	25,417	1,125.6
Mississippi	14,633	11,102	5,450	390.9	1,998	1,074	39.9	26,004	13,449	611.4	39,104	19,973	1,042.2
Missouri	8,787	34,082	17,421	1,020.4	7,835	4,075	24.3	75,604	39,716	1,256.8	117,521	61,212	2,301.5
Montana	17,042	9,572	5,897	172.2	4,874	2,822	91.6	22,913	14,391	387.8	37,359	23,110	651.6
Nebraska	14,904	19,328	10,152	853.0	7,725	3,579	89.5	22,900	12,308	719.1	49,953	26,039	1,661.6
Nevada	11,852	4,610	3,898	75.0	1,718	1,437	30.9	4,452	3,790	88.4	10,780	9,125	194.3
New Hampshire	6,978	2,572	1,294	16.0	514	257	2.6	7,311	3,782	45.9	10,397	5,333	64.5
New Jersey	26,099	9,741	4,339	53.4	4,715	2,354	5.3	28,596	13,324	43.2	43,052	20,017	101.9
New Mexico	9,441	4,188	2,643	50.8	3,187	2,028	57.2	9,442	6,251	181.7	16,817	10,922	289.7
New York	55,352	37,956	19,566	60.5	39,369	20,236	83.5	213,384	98,504	255.7	290,709	138,306	399.7
North Carolina	17,343	22,894	11,381	463.3	5,809	2,902	44.5	39,754	19,951	504.2	68,457	34,234	1,012.0
North Dakota	8,503	12,357	6,249	1,259.1	3,705	1,893	317.4	8,937	4,571	500.6	24,999	12,713	2,077.1
Ohio	28,497	41,720	21,458	147.6	29,373	15,917	54.5	64,124	30,242	84.0	135,217	67,617	286.1
Oklahoma	15,922	18,966	9,651	356.3	12,974	7,043	79.7	32,103	16,690	357.8	64,043	33,384	793.8
Oregon	5,610	6,411	3,604	55.6	5,378	3,317	32.1	19,239	12,038	203.8	31,028	18,959	291.5
Pennsylvania	34,421	14,318	7,592	44.7	64,041	32,695	76.7	78,627	39,925	291.2	156,986	80,212	412.6
Rhode Island	2,479	2,358	1,179	12.5	7,815	4,221	7.4	13,135	6,580	19.9	23,308	11,980	39.8
South Carolina	13,360	14,901	8,097	352.6	2,222	1,148	21.1	17,965	9,506	297.0	35,088	18,751	670.7
South Dakota	3,477	22,547	13,002	829.1	4,474	2,681	137.5	12,236	7,023	528.2	39,257	22,706	1,487.8
Tennessee	21,034	19,217	9,570	263.8	8,954	4,479	78.9	35,233	15,853	254.4	63,404	29,902	597.1
Texas	33,606	17,114	8,764	408.6	37,988	19,486	267.7	90,042	47,633	1,368.1	145,144	75,883	2,044.4
Utah	7,405	2,166	1,619	35.7	4,899	3,689	27.2	9,761	7,481	123.6	16,826	12,789	186.5
Vermont	4,543	3,361	1,683	15.2	479	239	1.6	8,259	4,321	87.2	12,099	6,243	104.0
Virginia	18,664	17,332	8,758	270.7	4,889	2,645	22.8	23,622	11,514	247.6	45,843	22,917	541.1
Washington	13,077	15,264	8,335	221.1	3,341	1,942	48.9	22,671	11,868	128.5	41,276	22,145	398.5
West Virginia	17,521	9,552	5,019	39.6	2,235	1,132	4.1	17,516	8,794	82.3	29,303	14,945	126.0
Wisconsin	18,646	14,679	7,772	137.4	3,301	1,732	43.1	44,818	22,348	392.7	62,798	31,852	573.2
Wyoming	3,673	8,353	5,462	163.4	242	156	10.1	13,252	8,695	237.6	21,847	14,313	411.1
Hawaii	5,065	1,830	915	3.1	4,503	2,233	5.9	4,385	1,972	11.0	10,718	5,120	20.0
District of Columbia	9,769	4,944	2,472	3.7	434	217	.9	8,875	3,965	2.7	14,253	6,654	7.3
Puerto Rico	11,909	4,533	2,203	25.9				17,350	7,976	63.4	21,883	10,179	89.3
TOTAL	761,446	766,185	408,263	12,702.7	455,201	242,371	2,711.5	1,681,716	860,196	16,317.1	2,903,102	1,510,830	31,731.3

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